WIND STRESS ANALYSIS IN TALL BUILDINGS

by

JOSEPH CHI-KIT LEE

B. S., Cheng Kung University, 1958
Tainan, Taiwan, China

A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1964

Approved by:

Reed F. Morse

Major Professor

TABLE OF CONTENTS

INTRODUCTION

The analysis of a tall building under vertical loads such as dead loads, live loads, and lateral loads due to wind is tedious and time consuming by any method. The distribution of wind pressure on a tall building is very complicated and irregular. In view of the many uncertainties involved, such as the exact pressure distribution, the torsion of the whole building frame, the secondary moments, an exact solution is apparently impossible.

Methods for analyzing wind stress can be found in some structural books, but no reference was made to the complete design of a tall building. Therefore, the writer of this report was doubtful about the manner in which a tall building was designed forty years ago when electronic computers were not available. Being interested in this problem, the author, within his knowledge, presents here a speedy and modified combination of several methods which can be outlined as follows:

First of all, the cantilever method is used to solve for the wind stresses which are combined with the stresses due to vertical live and dead loads in order to obtain the approximate size for columns and girders. (3), (7). Knowing the approximate sections in columns and girders the portal method is used for a more refined analysis of the wind stresses and the moment distribution method, which requires the approximate stiffness factors of all members, is used to recalculate the stresses due

---

Numbers in parentheses, thus: (3) (7), refer to corresponding items in the Bibliography.

to the vertical loads. The combination of the above two different kinds of stresses can be used to revise the size of columns and girders in the final design. (1), (8).

METHODS OF ANALYSIS OF WIND STRESS

In this analysis it is assumed that the steel frames of a tall building resists the entire wind load. Although masonry walls and partitions increase the lateral stability of the building, they are neglected in the analysis.

There are several methods that may be used to analyze the stresses in a tall building frame. A few of these are described briefly below:

The approximate methods--These methods are based on several assumptions and without consideration of the elastic behavior of members of the bent in the frame. These methods are quickly performed and, for buildings of typical proportions, are quite satisfactory. The resultant stresses may be somewhat in error. These methods ignore the secondary bending moments which result from changes in length of columns under direct wind stresses. For large proportions of bent height to width in the building, these moments may have considerable influence.

The principle approximate methods are:

(A) Cantilever method-----It is assumed that the points of contraflexure are located at the mid-point of girders and the mid-point of columns, the unit direct stresses vary as the distance of the columns from the center of gravity of the bent regardless of column area. (7)

(B) Portal method I-----The assumption for the location of the points of contraflexure is the same as that for the cantilever method. It is also assumed that the shear in each exterior column is the same and equals one-half the shear in an interior column. (7)

(C) Portal method II-----It is actually a modified method of both the cantilever method and the portal method I. It is assumed that the points of contraflexure are located at different heights according to the number of stories of the building. The amount of shear in the columns is equal to a factor which is related to the number of bays and columns times the total shear in the story. (7)

The above three methods are based on the assumption of horizontal shear which is arbitary. With equal spacing in columns, these methods can be used. But, for irregular column spacing and for tall buildings, errors may occur due to the relation of beams and columns.

(D) The Cross method-----This is the so called moment distribution method. For tall frames the large number of cycles necessary to effect an accurate solution renders it impractical. This method is more accurate than the cantilever and portal methods, but it cannot be applied if the sizes of beams and columns are unknown. (1)

(E) Witmer method of K percentages-----This method of K (Stiffness factor) percentage is based on the actual sizes and dimensions of the bent. For bents having a large variety of proportions moments were found to vary too much from results obtained by other methods to be considered permissible. (1) (8)

(F) Slope deflection method-----The method requires the solution for many unknowns and is impractical unless a computer is available. The slope deflection is the best known of the theoretical methods. The stresses found by the slope deflection method are not exact because the assumptions are not strictly valid.

From the above brief discussion, the following conclusion can be made:

If a preliminary design of a frame is made with an assumption of discontinuous girders, and if the secondary moments in the columns due to axial distortion caused by direct wind stress are neglected, the cantilever and portal methods can be used in analysis and design with some assumptions.

PROPOSED METHOD OF ANALYSIS IN THE DESIGN OF A TALL BUILDING

The analysis of multiple story building frames under vertical loads and wind loads by means of any method is admittedly tedious and time consuming. In this paper, a combination of several methods is presented.

The proposed method of design is demonstrated by the analysis of a 4-bay, 40-story building frame. To date, no simple exact method has been published. The purpose of this report is to simplify the method of analysis. The following procedures and assumptions can be found in the books on the theory of structures list in the bibliography:

(1) All columns and all girders in any story have the same sizes and cross sections. In the approximate design, the convenient method is used to obtain the approximate sizes of columns and girders. For simplifying the procedure, typical stories are taken as the subframes of a tall building, and the size of columns and girders are determined from the design manual of AISC Specification.

(2) By the cantilever method, the wind moments and direct stresses in columns can be determined by the weight of the whole structure.

(3) The approximate stresses are obtained by combining the stresses due to vertical and lateral wind forces.

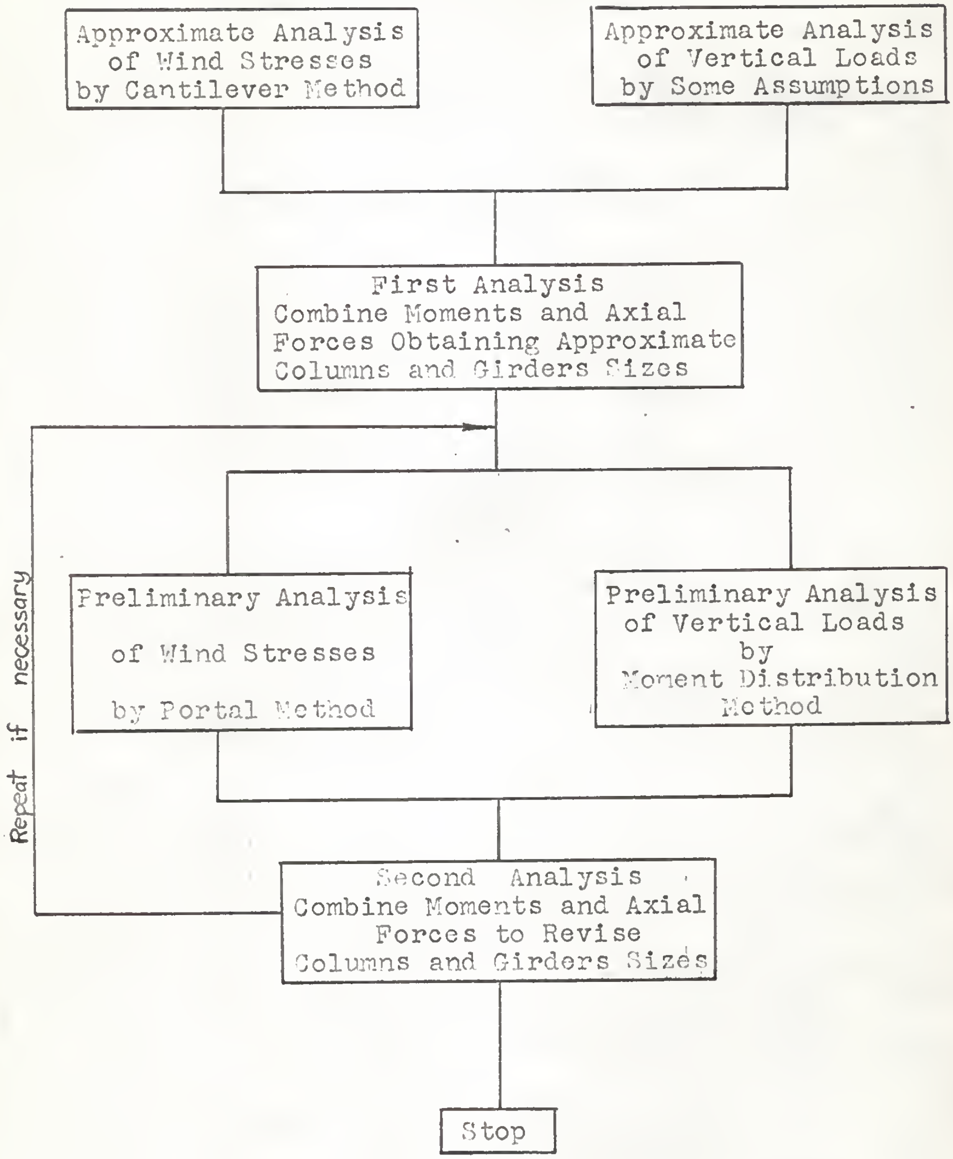
(4) After obtaining the stresses and approximate sizes of columns and girders, one can easily calculate the stiffness factors of all members.

(5) By applying the moment distribution method, to obtain the stresses due to vertical loads, and the portal method, for wind stresses, these stresses may be combined and the columns and girders selected. The typical floors are used in this operation.

The procedure of analysis of a tall building is demonstrated by analyzing a 4-bay, 40-story building frame with 12 feet height for each floor. The total height is 480 feet. There are six unknowns for each story, namely five joint rotations, θ, and one sidesway, R, and total of $6 \times 40 = 240$ unknowns in the entire building. The method (5) above can be used without the help of a computer.

The procedure is outlined in the following chart:

The analysis of a tall building frame would normally be accomplished by a "guess and check" procedure. Moments of inertia or stiffness factors could be approximated in the first analysis by using the AISC Specification. These approximate stiffness factors are the basis for the second analysis from which a set of revised bending moments are obtained for the whole frame. From the revised moments, a set of revised or final sizes in the frame can be obtained. The details of analysis will be discussed in the following sections.


Approximate Analysis of Wind Stresses by Cantilever Method
Approximate Analysis of Vertical Loads by Some Assumptions
First Analysis Combine Moments and Axial Forces Obtaining Approximate Columns and Girders Sizes
Preliminary Analysis of Wind Stresses by Portal Method
Preliminary Analysis of Vertical Loads by Moment Distribution Method
Repeat if necessary
Second Analysis Combine Moments and Axial Forces to Revise Columns and Girders Sizes
Stop

THE FIRST ANALYSIS

Wind forces become more important as the height of a structure is increased. The subject of wind resistance in tall buildings may be divided into three parts:

(A) The wind pressure must be assumed.

(B) The stresses due to the wind pressure should be determined.

(C) The working stresses should be determined.

A. Wind pressure

Most high buildings are in cities in which the maximum wind pressure to be resisted is specified by the municipal building code. New York City uses 30 lbs. per sq. ft. as the wind pressure for tall buildings. But the effect of walls, partitions and floors increases the rigidity of the frame, so that 20 lbs. per sq. ft. which corresponds to a wind velocity of 80 miles per hr., is used as a uniformly-applied wind pressure on the building. (8)

B. Determination of wind stresses

In the approximate design, the AISC Code is used to select the approximate sizes of columns and girders. The cantilever method is used to determine the wind stresses. After that the portal method is applied in the second design with the aid of the simplified typical floors.

C. Working stresses

Because wind loads seldom reach their maximum, greater working stresses are permissible for the wind loading acting alone or in combination with dead and live loads. The AISC Specification recommends that working stresses may be increased 33.33% above the allowable stress when members are subject to both wind loads and vertical loads, provided the required section computed on this basis is not less than that required for the design dead and live loads, computed without the one-third allowable stress increase.

(A) GENERAL LAYOUT

Floor loads and wind loads are assumed

Dead load (vertical)	Roof	100 lbs per sq. ft.
	Floor	150 lbs per sq. ft.
Live load (vertical)	Roof	25 lbs per sq. ft.
	Floor	50 lbs per sq. ft.
Wind load (horizontal)		20 lbs per sq. ft.

Working stresses for steel (AISC Code)

Tension f_s = 20,000 lbs per sq. in.

Compression f_s = varies with $K\frac{l}{r}$

Wind working stresses increase by one-third allowable stress

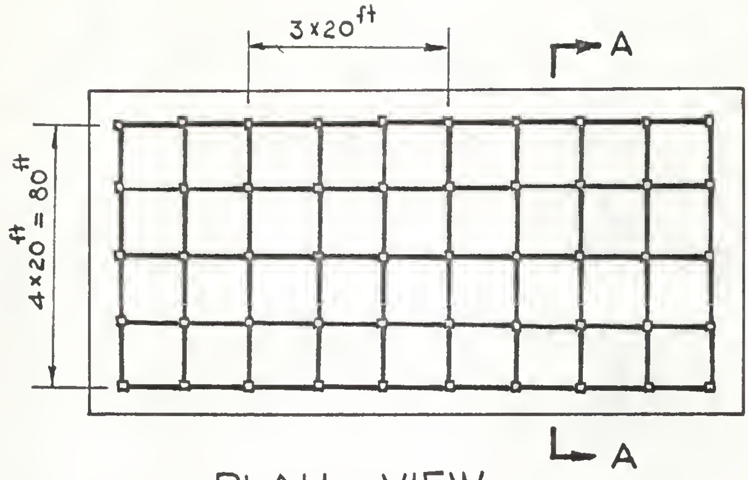
where K = Effective length factor, in this problem, it is equal to 1.0 according to the Commentary of AISC Specification. p 5-117

r = Governing radius of gyration

l = Span length

Assuming 12 feet of height for each floor and 6 feet for parapet, the building has 20 feet for the span between columns in both directions in this problem.

By the assumption of a 6 foot parapet, we have a uniform wind force of 4.8 kips at each floor. The first step is to compute the story shears and story moments of the columns due to the wind force. The 30th story shear is illustrated in Fig. 2, and the story moment at that story is illustrated in Fig. 3. For simplification, here, the story moment is equal to the product of story height 12 feet and the story shear H. This moment is a little bit larger than the story moment computed from cantilever method by 5 percent and that is considered acceptable. These story shears and moments at 40th, 30th, 20th, 10th, 2nd stories are computed as illustrated



PLAN VIEW

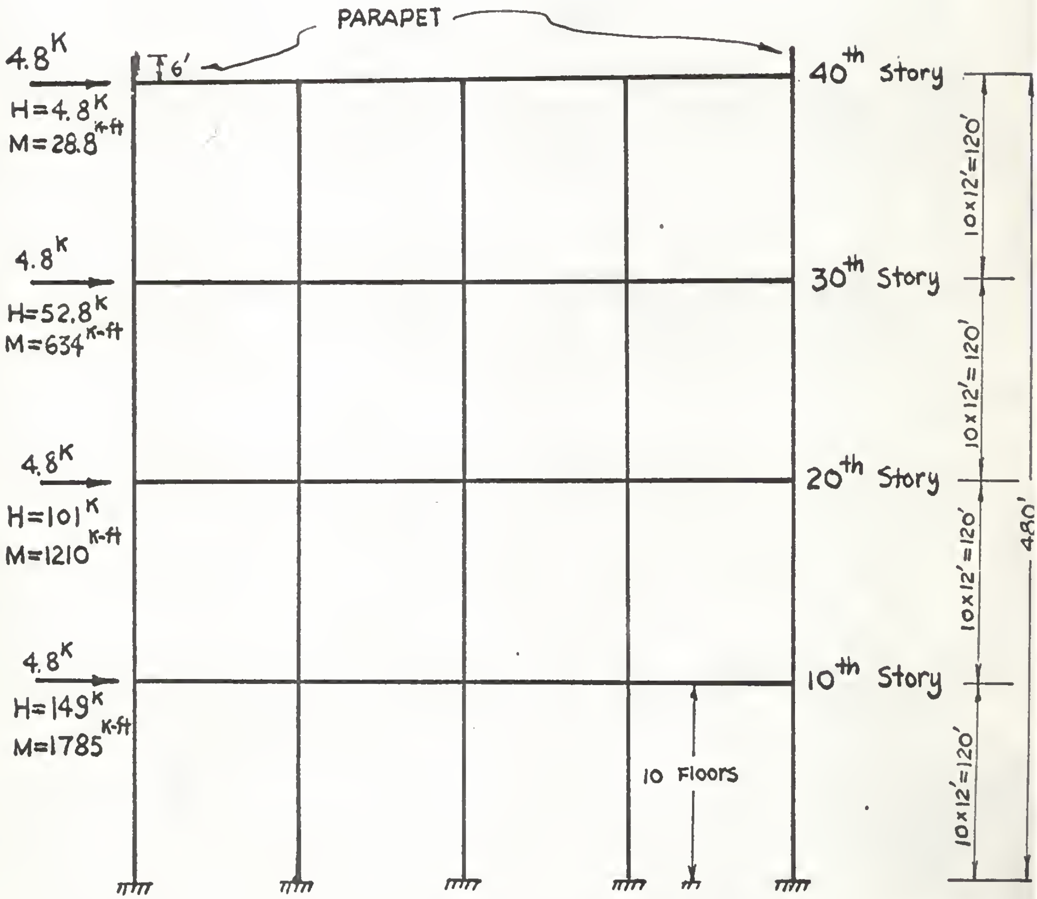


FIG. 1 SECTION A-A

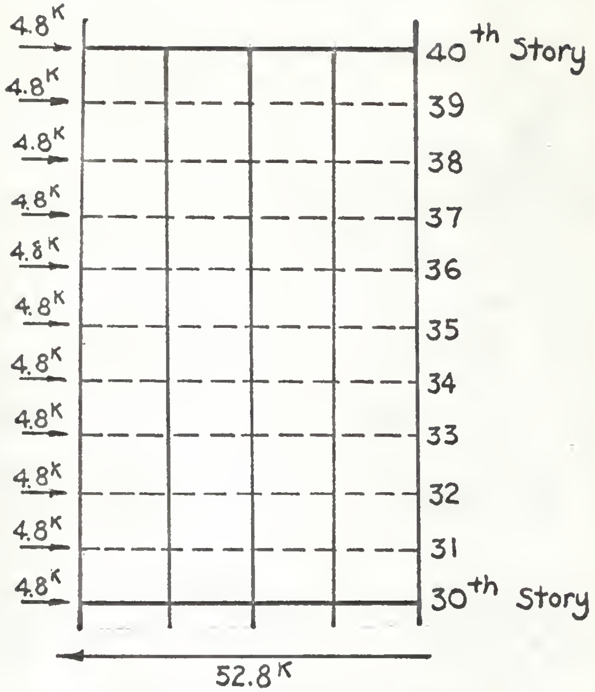
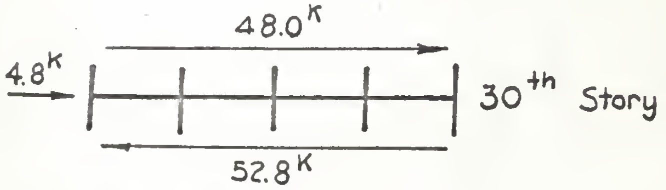


FIG. 2 Story Shear in Columns between 30th and 29th Story



BY Cantilever Method

Story Moment $= 48^K \times 6^{ft} + 52.8^K \times 6^{ft} = 604.8^{K\text{-}ft}$

For Simplification

Story Moment $= 52.8^K \times 12^{ft} = 633.6^{K\text{-}ft}$

$$100 \times \frac{633.6 - 604.8}{604.8} = 4.76 \doteqdot 5\,\%$$

FIG. 3 Story Moment in Columns in 30th Story

1	2	3	4	5	
Wind Load		Vertical Loads per Column			
Shear	Moment	Dead Load	Live Load	Total Axial Loads	
4.8^k	$28.8^{k\text{-}ft}$	$100 \times 400^{ft^2} = 40^k$	$25 \times 400\, ft^2 = 10^k$	50^k	40^{th} story
52.8^k	$634.0^{k\text{-}ft}$	$150 \times 400 = 60^k$ $40 + 10 \times 60 = 640^k$	$50 \times 400 = 200^k$ $10 + .6 \times 200 = 130^{k*}$ $10 + 10 \times 20 = 210^k$	770^k	30^{th} "
101^k	$1210^{k\text{-}ft}$	$640 + 10 \times 60 = 1240^k$	$130 + 120 = 250^{k*}$ $210 + 10 \times 20 = 410^k$	1490^k	20^{th} "
149^k	$1785^{k\text{-}ft}$	$1240 + 10 \times 60 = 1840^k$	$250 + 120 = 370^{k*}$ $410 + 10 \times 20 = 610^k$	2210^k	10^{th} "
187.2^k	$2250^{k\text{-}ft}$	$1840 + 8 \times 60 = 2320^k$	$370 + 96 = 466^{k*}$ $610 + 8 \times 20 = 770^k$	2786^k	2^{nd} "

Table 1 Wind Shear, Wind Moment, Dead Load, Live Load and Reduced Live Load (*) Combintion Dead Load for the Typical Stories

in column 1 and 2 in the Table 1. The axial forces due to dead and live loads are in column 3 and 4. Total column axial forces due to dead and reduced live load are shown in column 5. The reduction of live load is based on the American Standard Building Code Requirements, page 5-163.

(a) No reduction on roof live load.

(b) For live load of 100 lbs. per sq. ft. or less, the design live load on any member supporting 150 square feet or more, the reduction shall exceed neither R as shown in the following formula nor 60 percent.

$$R(\%) = 100 \times \frac{D + L}{4.33L} = 100 \times \frac{150 + 50}{4.33 \times 50} = 100 \frac{200}{216.5} = 93.6\% > 60\%$$

in which

$R(\%)$ = reduction of what (of live load) in percent

D = dead load per square foot of area supported by the member

For live load exceeding 100 lbs per sq. ft., no reduction shall be made, except that the design live loads on columns may be reduced 20%. In this problem, 60 percent is used as the reduction factor.

(B) PRIMARY CONSIDERATIONS

From the above Table 1, the dimensions of columns can be determined by using the AISC Code.

For the approximate dimensions for all girders, the uniform vertical loads acting on each girder is

$(100 + 25) \times 20 = 2{,}500$ lbs per ft. For roof at 40th story

$(150 + 50) \times 20 = 4{,}000$ lbs per ft. For 30th, 20th, ... stories

These girders will be rigidly connected to the columns, but they will not be entirely fixed. Hence, we can make the first design on the basis of a controlling moment of $\frac{1}{10}WL^2$ (3).

(C) SELECTION OF APPROXIMATE DIMENSIONS

(1) Columns

Columns are designed from the top downward based on the dead and reduced live loads. The dimensions shown in Table 1 are designed by AISC Code. The column size will be considered constant near the typical stories. Wind bending moments are reduced to an equivalent central load from the relation (3)

$$P = \frac{M}{S/A}$$

The factor S/A varies but slightly for the 14 x 16 WF sections being from 5.4 to 5.6 only. Its variation is always small. Hence, 14 x 16 WF sections will be chosen for the whole building.

For 40^{th} story

D.L. L.L. $= 50^k$

Moment reduced to equivalent force $\frac{28.8 \times 12}{5.5 \times 5} = 12.5^k$

$\overline{}$

62.5^k

Try 14WF43 $F_y = 36^{ksi}$ $L/r = 12 \times 12/1.89 = 76.1$ $P = 200^k$

For 30^{th} story

D.L. L.L. $= 770^k$

Moment reduced to equivalent force $\frac{634 \times 12}{5.5 \times 5} = 276^k$

$\overline{}$

1046^k

1046^k does not include the total column weight above 30^{th} story, so try 14WF167 $F_y = 42^{ksi}$. This section has the ability to support 1201^k.

In the same way, we have the required approximate column sections in 20^{th}, 10^{th}, and 2^{nd} stories. It is shown in Fig. 2.

(2) Girders

For the girder sections, the moments are:

$$M = \frac{1}{10} \times 2500 \times (20)^2 \times 12 = 1,200,000 \text{ lbs in} \qquad \text{For roof}$$

$$S = M/f_s = 1,200,000/20,000 = 60^{in3}$$

The required section modulus is furnished by a 14WF43 $F_y = 36^{ksi}$ (S = 62.7)

$$M = \frac{1}{10} \times 4,000 \times (20)^2 \times 12 = 1,920,000 \text{ lbs in} \qquad \text{For other floors}$$

$$S = M/f_s = 1,920,000/20,000 = 96^{in3}$$

The required section modulus is then furnished by a 16WF64 $F_y = 36^{ksi}$. (S = 104.2)

All approximate sections are shown in Fig. 4.

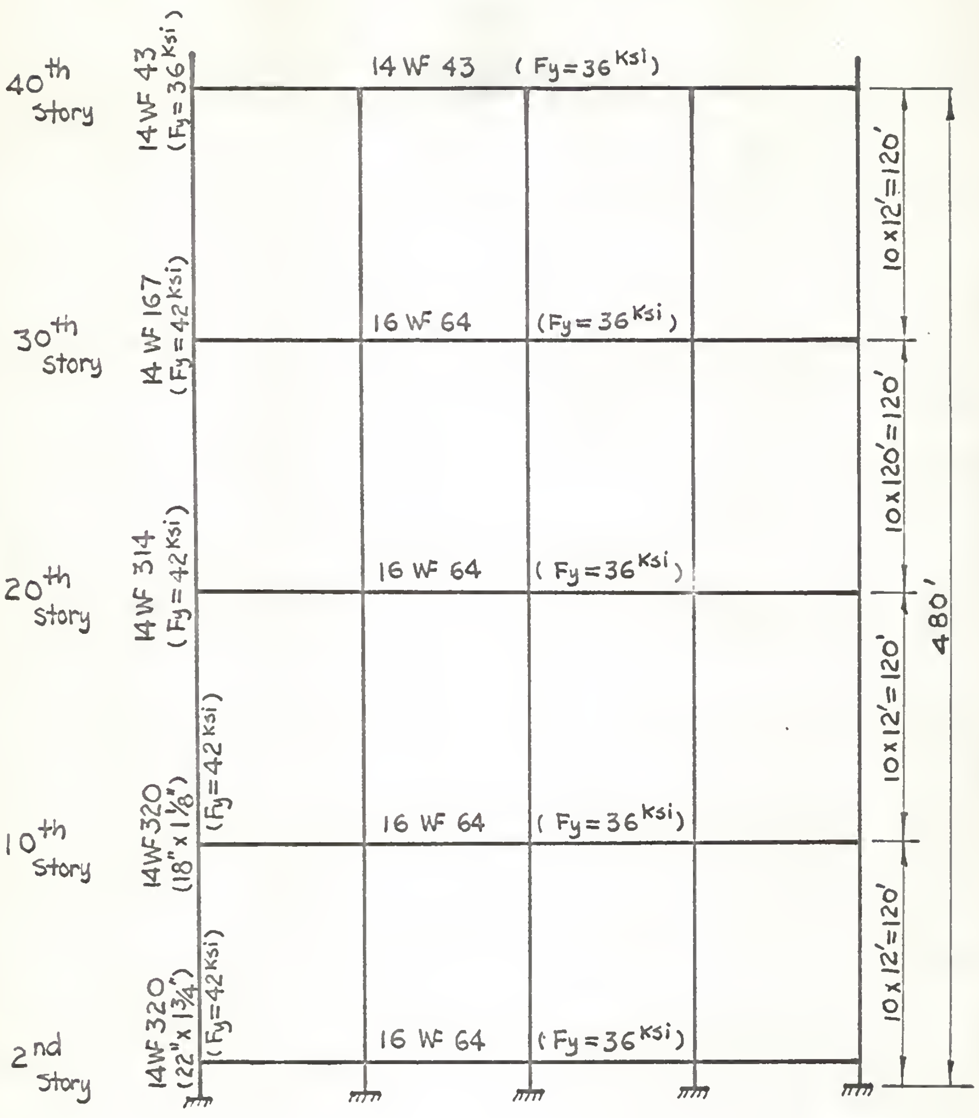


FIG. 4 Approximate Column and Girder Sizes for Use in First Analysis with Vertical Loads and Wind Force

SECOND ANALYSIS

(A) WIND LOAD STRESSES ANALYZED BY PORTAL METHOD (8)

The portal method for computing approximate moments, shears, and axial forces from the horizontal force due to wind loads is based on the following assumptions:

(1) The point of contraflexure of each column is at mid-height of the story.

(2) The point of contraflexure of each girder is at its mid-point.

(3) The horizontal shear on any frame is divided equally among the members of the panel. An outer column takes but one-half of the shear of an interior column.

(4) The wind load is resisted entirely by the steel frame.

The actual computations for this method are extremely simple. At the inflection points, $M = 0$, only shears and axial forces are transmitted from one-half to the other. An inflection point is equivalent to a hinge, which transfers only forces but not moments. All moments, shears and axial forces are computed by statics shown in Fig. 5.

For illustration, consider points C and D, the total shear in the section of 30^{th} story is 52.8^{k}. The shear in each exterior column is

$52.8/8 = 6.6^{k}$

and in each interior column is

$6.6 \times 2 = 13.2^{k}$

The shears in other floors are obtained in the same way, and act at the hinges (inflection points) as shown.

How, considering the equilibrium of the rigid structure between hinges a, b, and c,

$$6.6 \times 6 = 39.6^{k\text{-}ft}$$

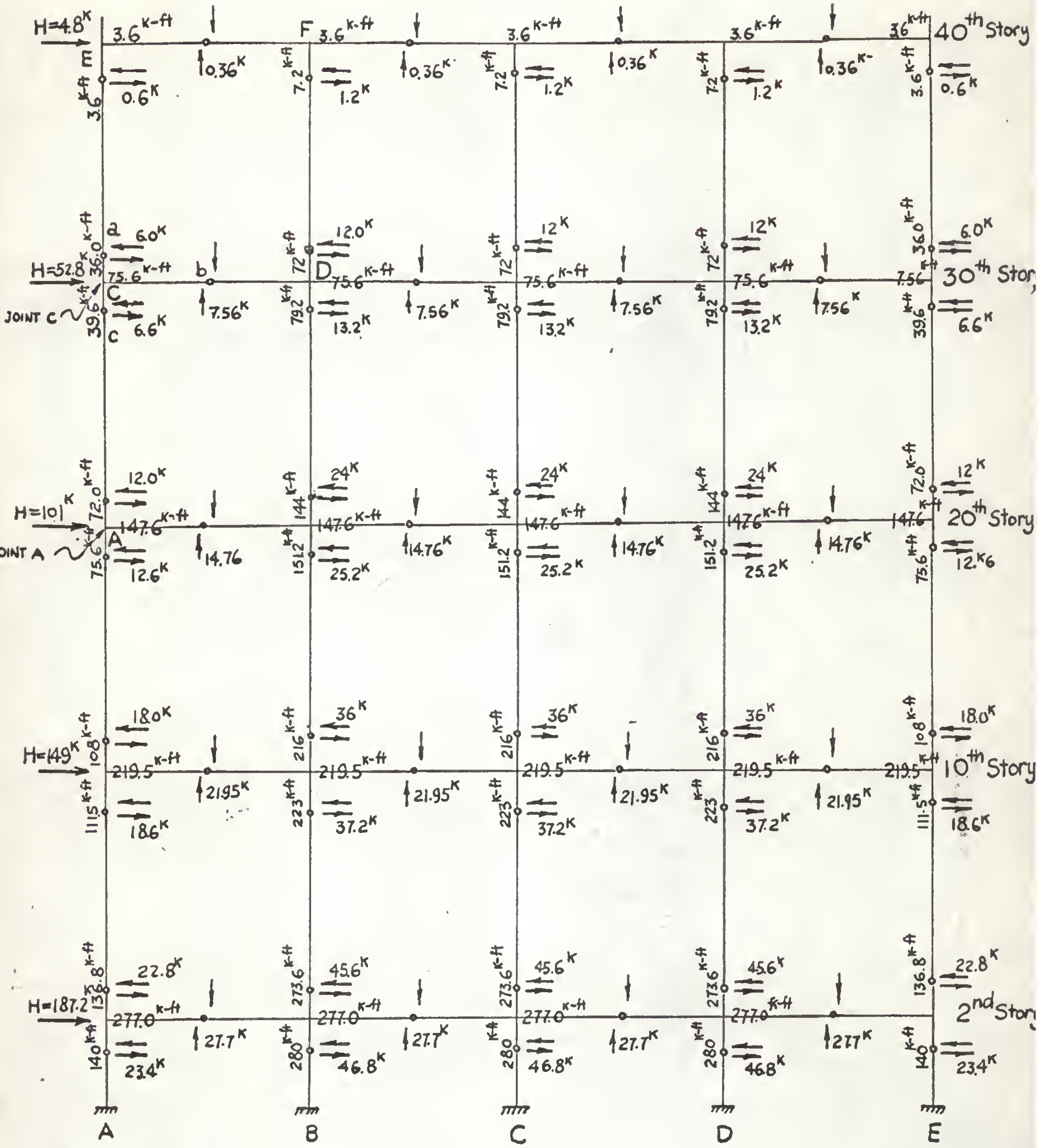


FIG. 5 Wind Stresses Analysis by Portal Method

The column moment is obtained directly by multiplying the shear times its lever arm, 6 feet. The girder moment at C, for equilibrium, is equal and opposite to the sum of the columns moments. The shear in the girder is obtained by recognizing that its moment (shear times the half span of girder) must be equal to the girder moment at C. Hence, this shear is

$$(6.6 \times 6 + 6.0 \times 6)/10 = 7.56^{k}$$

The moment at the other end D, is equal to the moment at C. At D, the column moments are computed in the same way from knowing the shears and lever arms. The girder moment to the right of C is

$$7.56 \times 10 = 75.6^{k\text{-}ft.}$$

Axial forces in the columns are also obtained by statics. After obtaining shears at every joint, the axial force can be computed by equilibrium.

(B) ANALYSIS OF VERTICAL LOADS STRESSES (8)

The approximate wind stresses were computed by the portal method. For analysis of the vertical live and dead load stresses, the live load must be arranged in a great variety of different schemes. Such schemes are required to obtain the maximum moments in all members. The following checkerboard patterns will result in larger effects than others.

A simplified approximate moment distribution method which allows the determination of these moments with reasonable accuracy and a limited amount of computation is presented here. In this method, moments are determined with sufficient accuracy by breaking up the entire frame into simpler typical subframes as shown in Fig. 6.

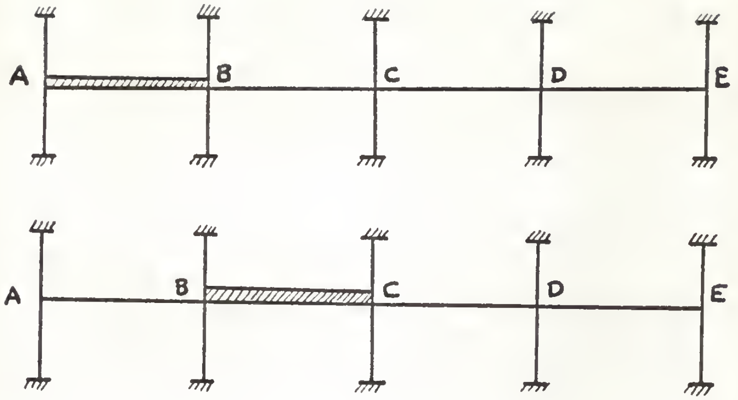


Fig. 6.

Each of these consists of one girder, plus the top and bottom columns framing into that particular girder. In this problem, considerable simplification can be achieved by noting that:

(1) All live and dead loads are uniformly distributed although of varying magnitudes.

(2) The structure is symmetrical.

To reduce computations, moments are determined separately for a uniform load of 1 kip per ft. placed individually on span AB and BC. The moments for all other loadings shown in Fig. 7 can be computed by simple superposition of the moments obtained from these two loading conditions. Also, simplifications such as these depend on the shape of the structure and the type of loading and should be used when possible.

The fixed-end moments $WL^2/12$ for W=1 kip per ft. are $33.3^{k\text{-}ft}$. The stiffnesses and distribution factors are computed in the usual manner from the given lengths and moment of inertia. For this particular type of moment computation in building frames, a special way of recording the calculations

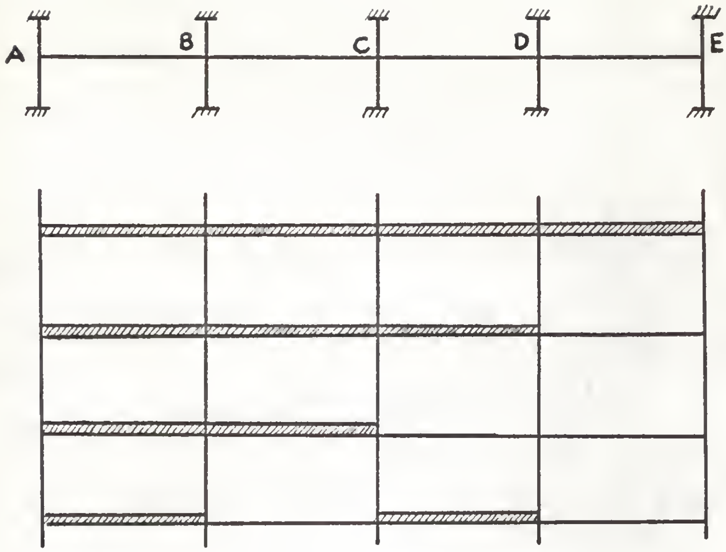


Fig. 7. Determination of moments due to live load in building frames.

is often found convenient. This scheme is shown in detail in Figs. 8-17 and is self-explanatory. It has the advantage that all figures are written in one instead of two directions. Carry-overs along the girders can be indicated by arrows in the usual manner. By taking advantage of the symmetry of the structure, it is seen that all component moments on one side are equal and opposite to those for the symmetrically located sections on the other side. The resulting moments caused by unit loads on individual spans of each story are shown in Tables 2-6.

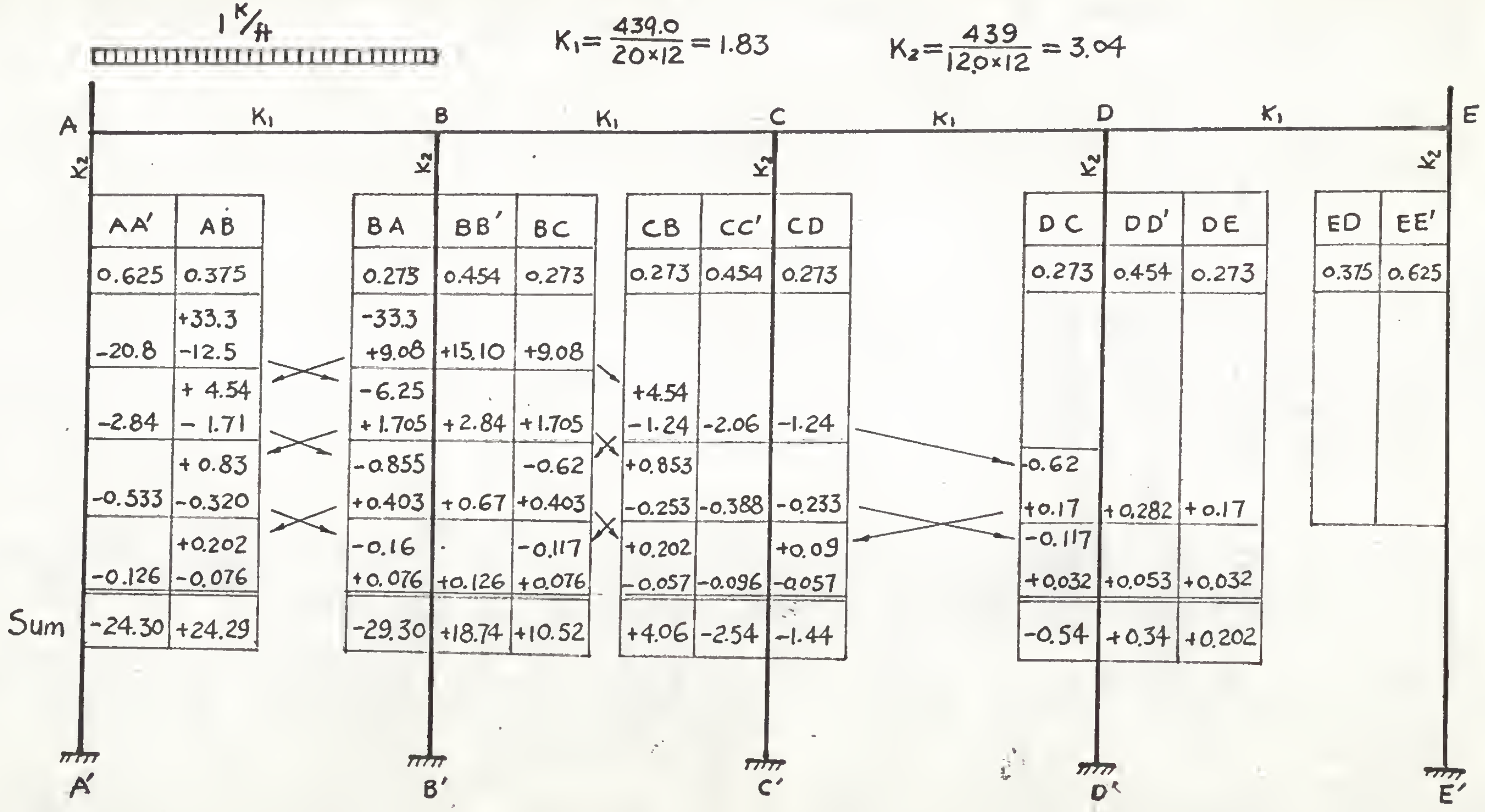
The moments for span loads on CD and DE are equal and opposite to those for symmetrically located sections for span loads on BC and AB respectively.

(C) THE COMBINATION OF MAXIMUM MOMENTS DUE TO WIND AND VERTICAL LOADS

From these moments caused by the unit loads, the actual maximum frame moments due to dead and live loads (from Tables 2-6) can be combined with the wind load moments (from Fig. 2 by portal method).

$$FEM = \frac{1}{12} WL^2 = \frac{1}{12}(1) \times (20)^2 = 33.3^{\,K\text{-}ft}$$

$$K_1 = \frac{439.0}{20 \times 12} = 1.83 \qquad K_2 = \frac{439}{120 \times 12} = 3.04$$



	AA'	AB
	0.625	0.375
		+33.3
	-20.8	-12.5
		+ 4.54
	-2.84	- 1.71
		+ 0.83
	-0.533	-0.320
		+0.202
	-0.126	-0.076
Sum	-24.30	+24.29

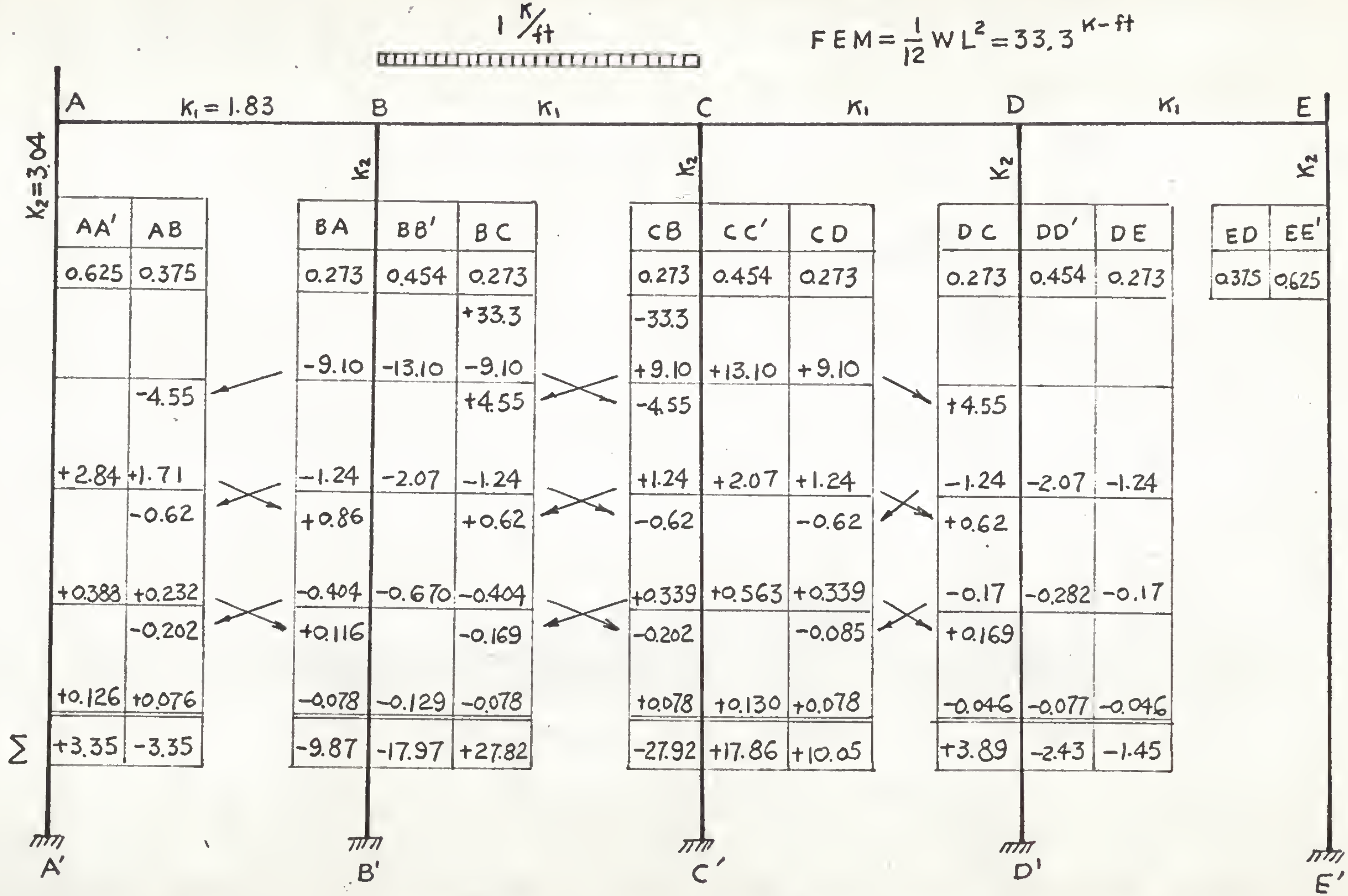
BA	BB'	BC
0.273	0.454	0.273
-33.3		
+9.08	+15.10	+9.08
-6.25		
+1.705	+2.84	+1.705
-0.855		-0.62
+0.403	+0.67	+0.403
-0.16		-0.117
+0.076	+0.126	+0.076
-29.30	+18.74	+10.52

CB	CC'	CD
0.273	0.454	0.273
+4.54		
-1.24	-2.06	-1.24
+0.853		
-0.253	-0.388	-0.233
+0.202		+0.09
-0.057	-0.096	-0.057
+4.06	-2.54	-1.44

DC	DD'	DE
0.273	0.454	0.273
-0.62		
+0.17	+0.282	+0.17
-0.117		
+0.032	+0.053	+0.032
-0.54	+0.34	+0.202

ED	EE'
0.375	0.625

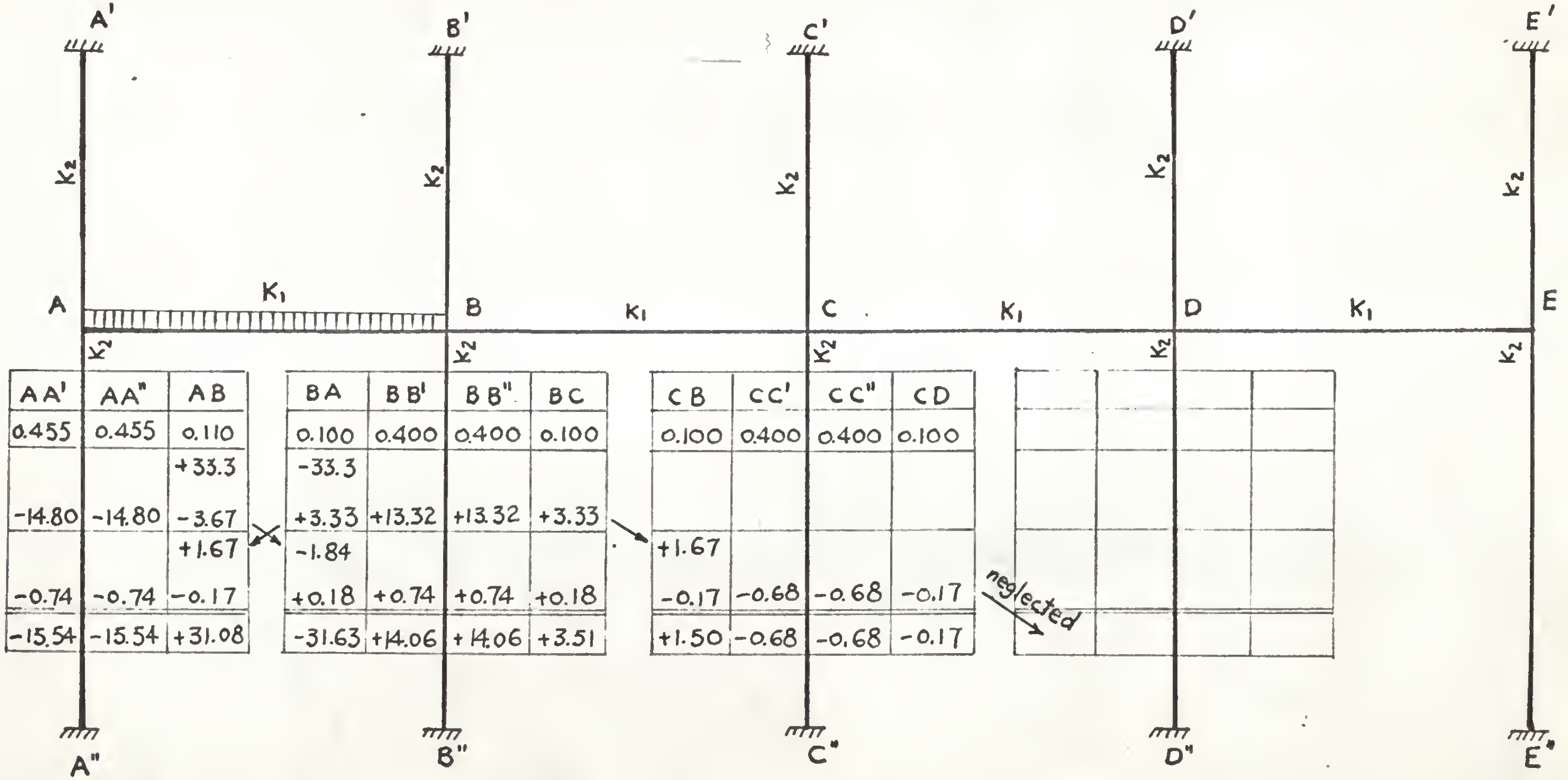
FIG. 8 The 40th Story



	AA'	AB	BA	BB'	BC	CB	CC'	CD	DC	DD'	DE	ED	EE'
	0.625	0.375	0.273	0.454	0.273	0.273	0.454	0.273	0.273	0.454	0.273	0.375	0.625
					+33.3	-33.3							
			-9.10	-13.10	-9.10	+9.10	+13.10	+9.10					
		-4.55			+4.55	-4.55			+4.55				
	+2.84	+1.71	-1.24	-2.07	-1.24	+1.24	+2.07	+1.24	-1.24	-2.07	-1.24		
		-0.62	+0.86		+0.62	-0.62		-0.62	+0.62				
	+0.388	+0.232	-0.404	-0.670	-0.404	+0.339	+0.563	+0.339	-0.17	-0.282	-0.17		
		-0.202	+0.116		-0.169	-0.202		-0.085	+0.169				
	+0.126	+0.076	-0.078	-0.129	-0.078	+0.078	+0.130	+0.078	-0.046	-0.077	-0.046		
Σ	+3.35	-3.35	-9.87	-17.97	+27.82	-27.92	+17.86	+10.05	+3.89	-2.43	-1.45		

FIG. 9 The 40^{th} Story

$$FEM = \frac{1}{12} WL^2 = 33.3^{k\text{-}ft} \qquad K_1 = \frac{833.8}{20 \times 12} = 3.48 \qquad K_2 = \frac{2020.8}{12 \times 12} = 14.1$$



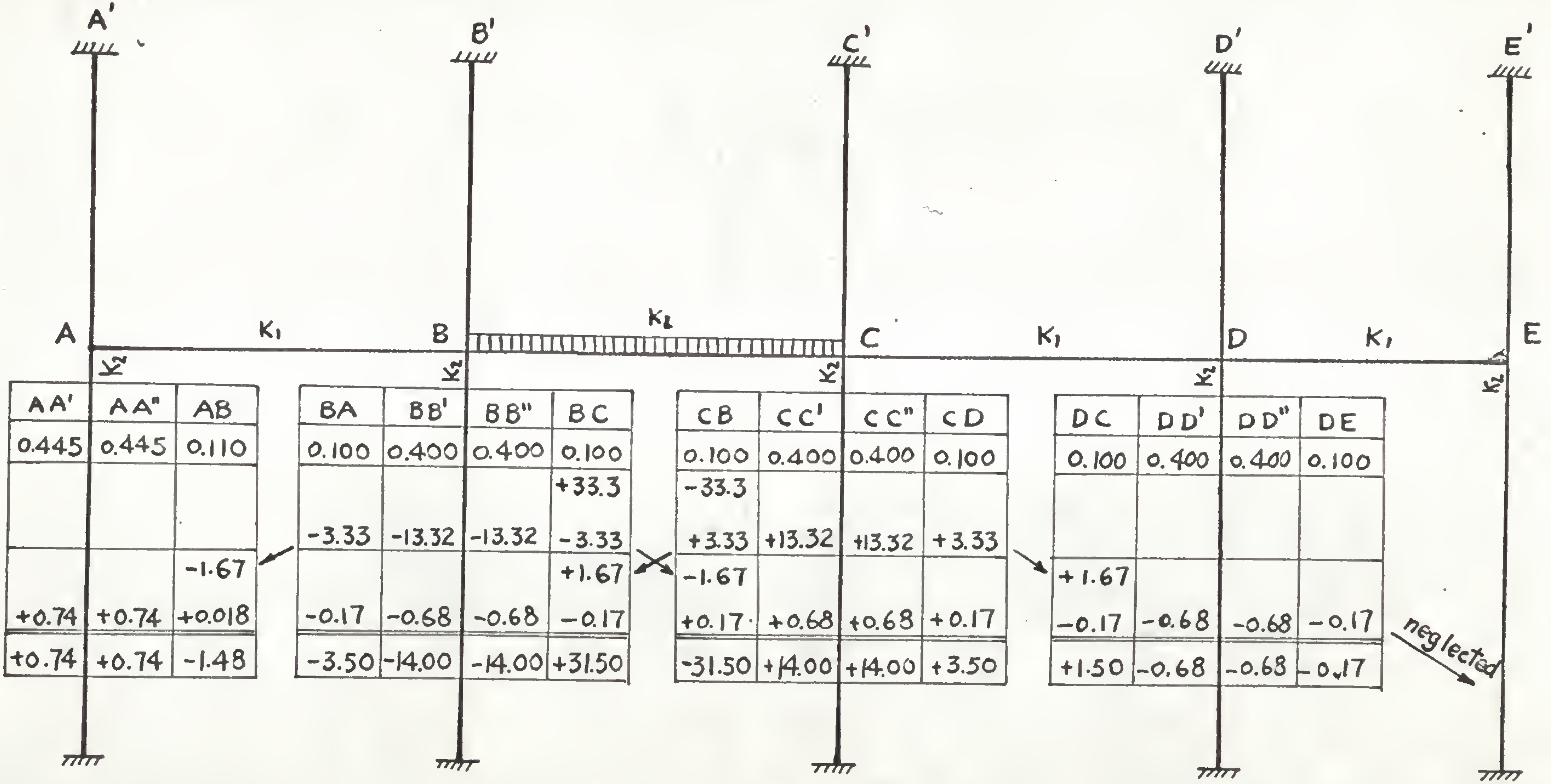
AA'	AA"	AB
0.455	0.455	0.110
		+33.3
-14.80	-14.80	-3.67
		+1.67
-0.74	-0.74	-0.17
-15.54	-15.54	+31.08

BA	BB'	BB"	BC
0.100	0.400	0.400	0.100
-33.3			
+3.33	+13.32	+13.32	+3.33
-1.84			
+0.18	+0.74	+0.74	+0.18
-31.63	+14.06	+14.06	+3.51

CB	CC'	CC"	CD
0.100	0.400	0.400	0.100
+1.67			
-0.17	-0.68	-0.68	-0.17
+1.50	-0.68	-0.68	-0.17

FIG. 10 30th Story

$FEM = \frac{1}{12} WL^2 = 33.3^{k-ft}$ $K_1 = 3.48$ $K_2 = 14.10$



AA'	AA"	AB
0.445	0.445	0.110
		-1.67
+0.74	+0.74	+0.018
+0.74	+0.74	-1.48

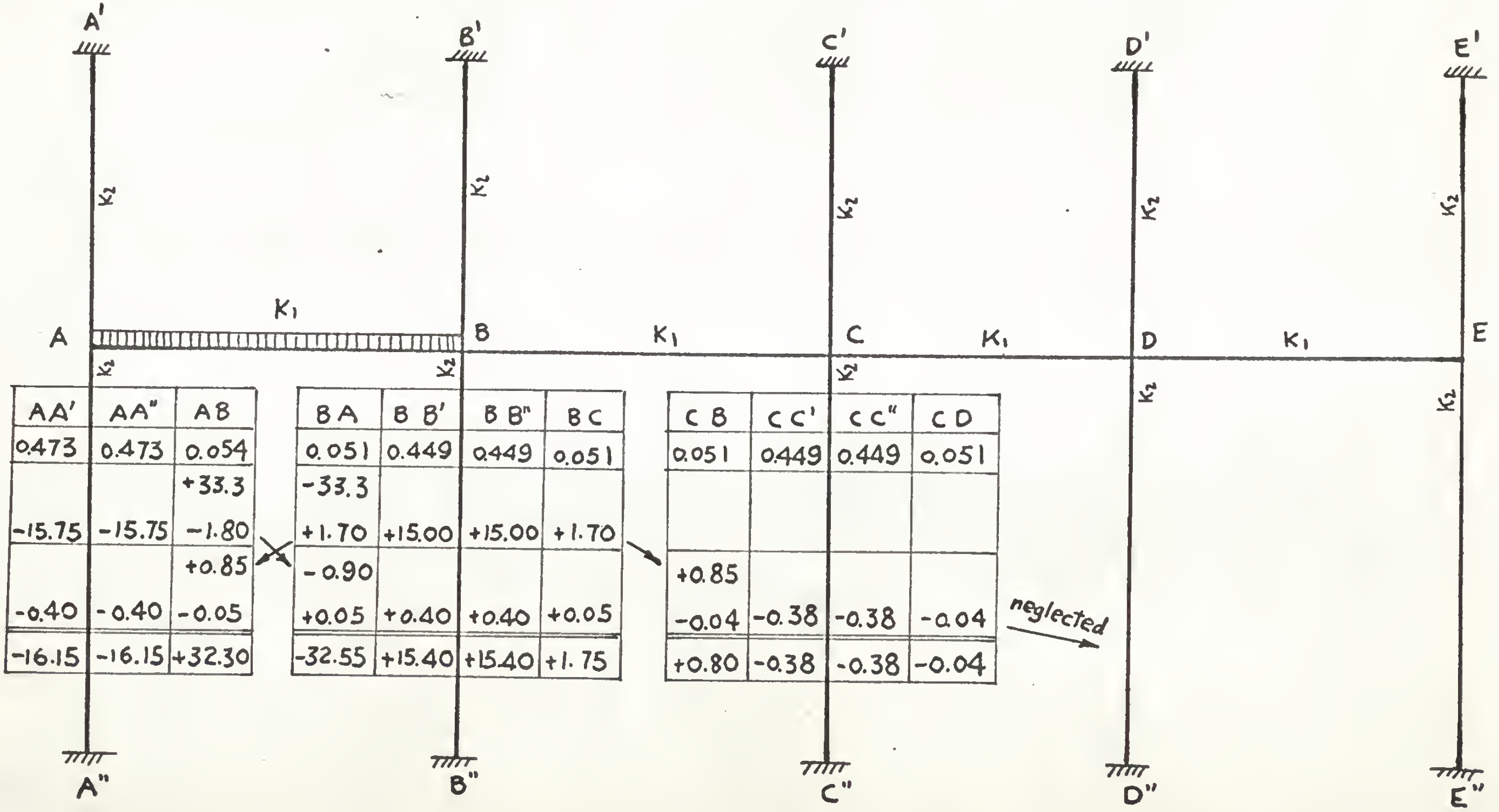
BA	BB'	BB"	BC
0.100	0.400	0.400	0.100
			+33.3
-3.33	-13.32	-13.32	-3.33
			+1.67
-0.17	-0.68	-0.68	-0.17
-3.50	-14.00	-14.00	+31.50

CB	CC'	CC"	CD
0.100	0.400	0.400	0.100
-33.3			
+3.33	+13.32	+13.32	+3.33
-1.67			
+0.17	+0.68	+0.68	+0.17
-31.50	+14.00	+14.00	+3.50

DC	DD'	DD"	DE
0.100	0.400	0.400	0.100
+1.67			
-0.17	-0.68	-0.68	-0.17
+1.50	-0.68	-0.68	-0.17

FIG. 11 The 30th Story

$$FEM = \frac{1}{12} WL^2 = 33.3^{K\text{-}ft} \qquad K_1 = 3.48 \qquad K_2 = \frac{4399.4}{12 \times 12} = 30.6$$



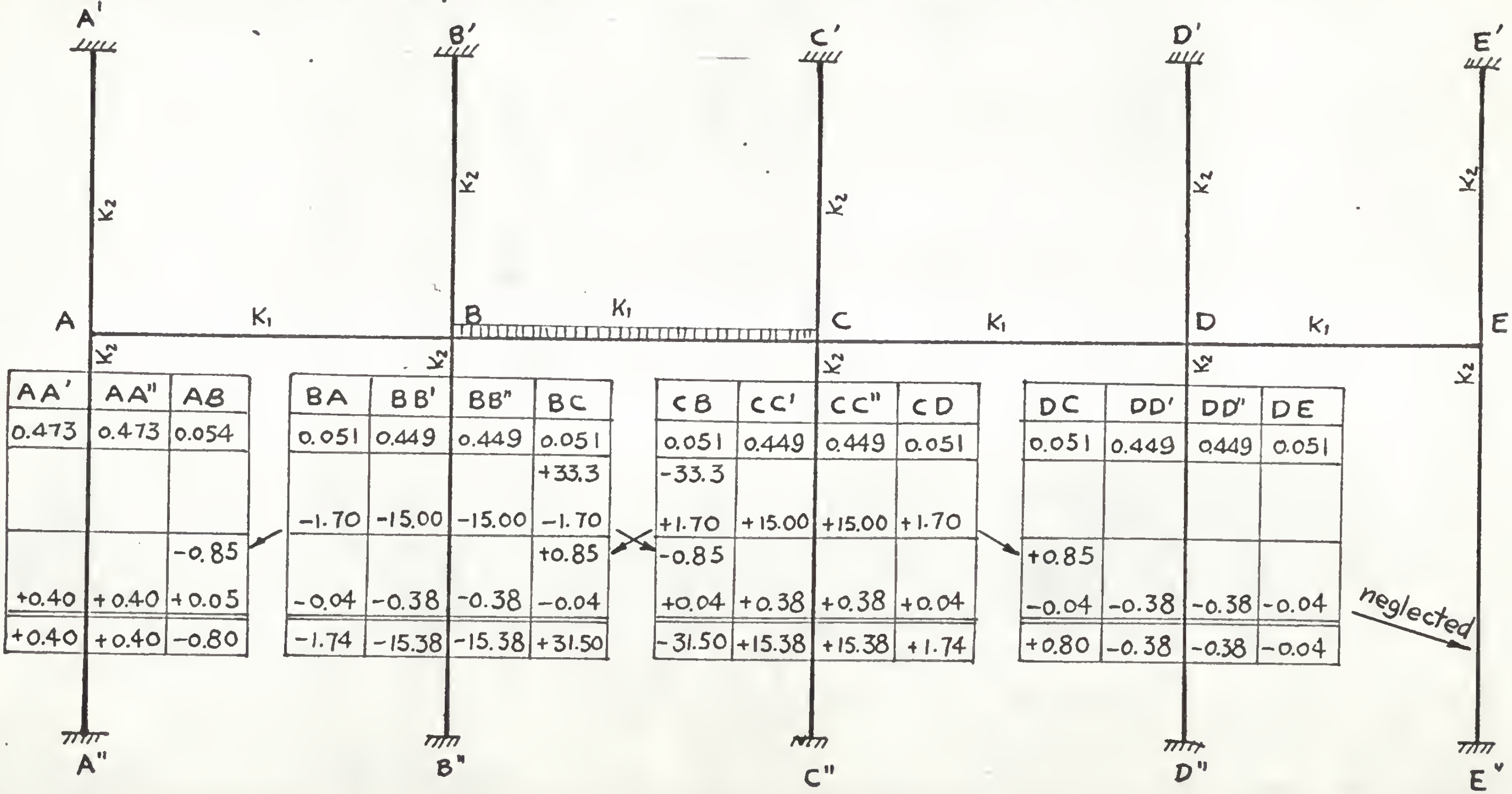
AA'	AA"	AB
0.473	0.473	0.054
		+33.3
-15.75	-15.75	-1.80
		+0.85
-0.40	-0.40	-0.05
-16.15	-16.15	+32.30

BA	BB'	BB"	BC
0.051	0.449	0.449	0.051
-33.3			
+1.70	+15.00	+15.00	+1.70
-0.90			
+0.05	+0.40	+0.40	+0.05
-32.55	+15.40	+15.40	+1.75

CB	CC'	CC"	CD
0.051	0.449	0.449	0.051
+0.85			
-0.04	-0.38	-0.38	-0.04
+0.80	-0.38	-0.38	-0.04

FIG. 12 The 20^{th} Story

$FEM = \frac{1}{12} WL^2 = 33.3^{k\text{-}ft}$ $K_1 = 3.48$ $K_2 = 30.6$



AA'	AA"	AB
0.473	0.473	0.054
		-0.85
+0.40	+0.40	+0.05
+0.40	+0.40	-0.80

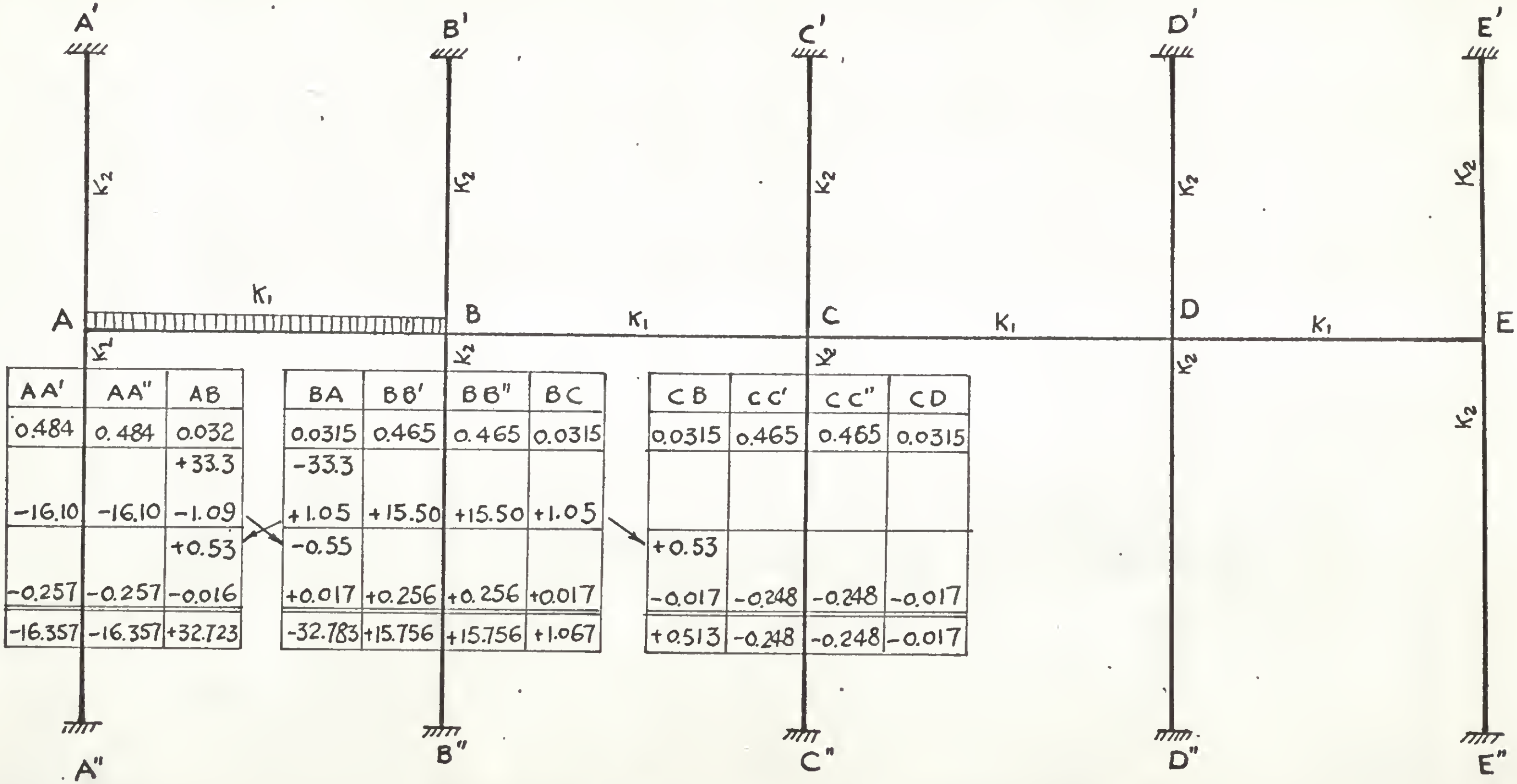
BA	BB'	BB"	BC
0.051	0.449	0.449	0.051
			+33.3
-1.70	-15.00	-15.00	-1.70
			+0.85
-0.04	-0.38	-0.38	-0.04
-1.74	-15.38	-15.38	+31.50

CB	CC'	CC"	CD
0.051	0.449	0.449	0.051
-33.3			
+1.70	+15.00	+15.00	+1.70
-0.85			
+0.04	+0.38	+0.38	+0.04
-31.50	+15.38	+15.38	+1.74

DC	DD'	DD'	DE
0.051	0.449	0.449	0.051
+0.85			
-0.04	-0.38	-0.38	-0.04
+0.80	-0.38	-0.38	-0.04

FIG. 13 The 20^{th} Story

$FEM = \frac{1}{12} WL^2 = 33.3^{K\text{-}ft}$ $\quad K_1 = 3.48 \quad K_2 = \frac{7403}{12 \times 12} = 51.4$



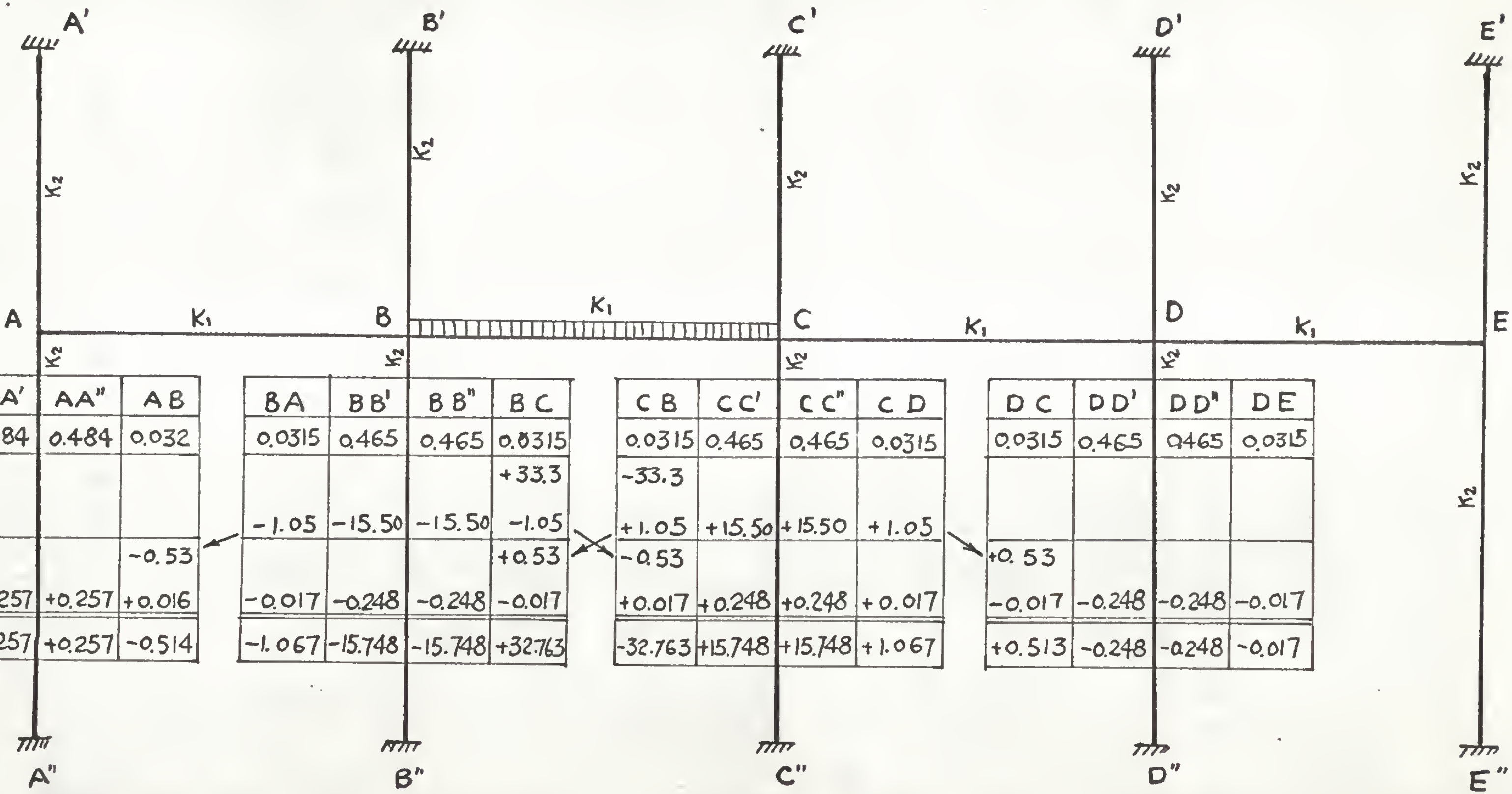
AA'	AA''	AB
0.484	0.484	0.032
		+33.3
-16.10	-16.10	-1.09
		+0.53
-0.257	-0.257	-0.016
-16.357	-16.357	+32.723

BA	BB'	BB''	BC
0.0315	0.465	0.465	0.0315
-33.3			
+1.05	+15.50	+15.50	+1.05
-0.55			
+0.017	+0.256	+0.256	+0.017
-32.783	+15.756	+15.756	+1.067

CB	CC'	CC''	CD
0.0315	0.465	0.465	0.0315
+0.53			
-0.017	-0.248	-0.248	-0.017
+0.513	-0.248	-0.248	-0.017

FIG. 14 The 10th Story

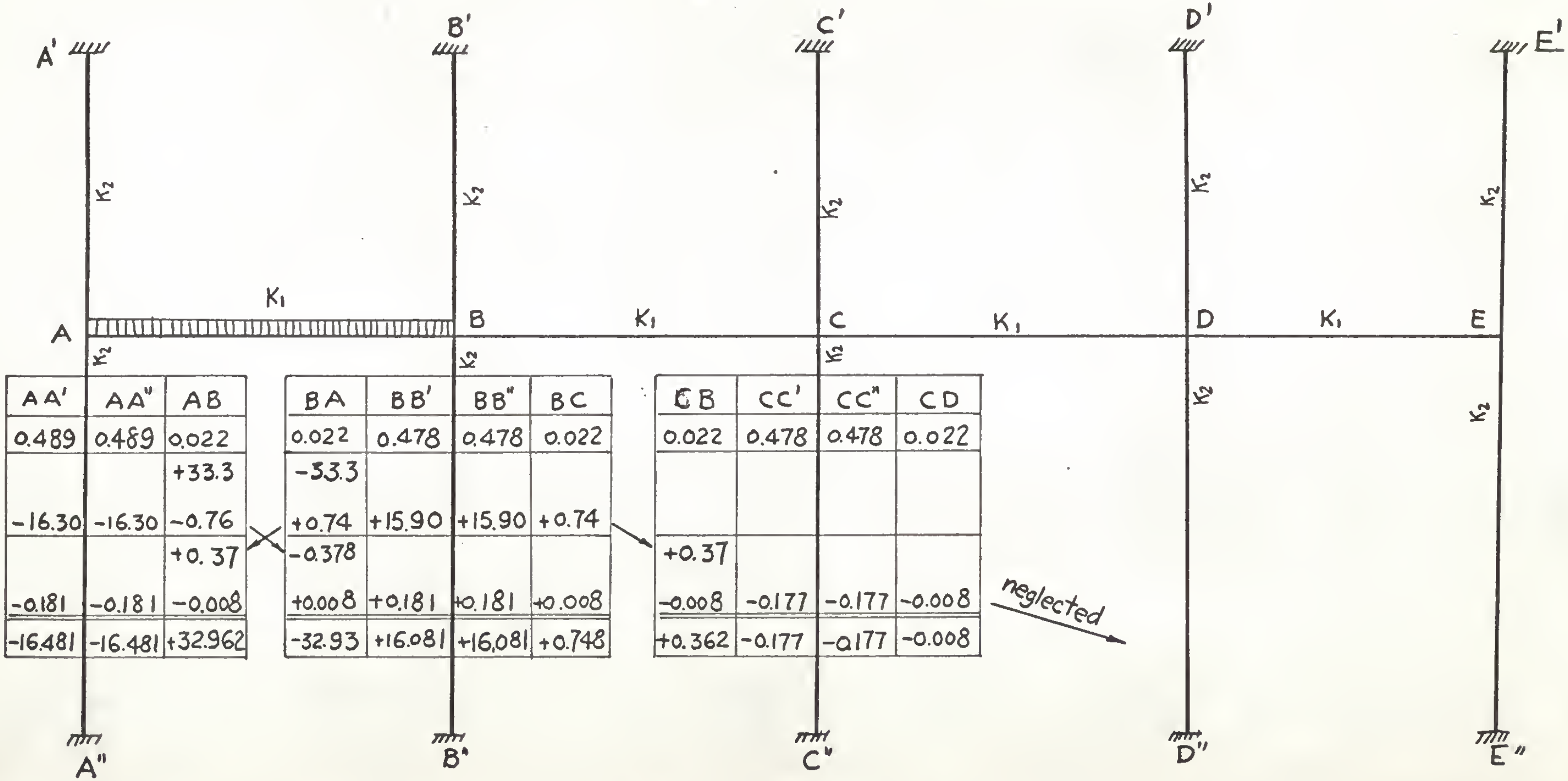
FEM = 33.3 $^{k-ft}$ $K_1 = 3.48$ $K_2 = 51.4$



	AA'	AA"	AB	BA	BB'	BB"	BC	CB	CC'	CC"	CD	DC	DD'	DD"	DE
	0.484	0.484	0.032	0.0315	0.465	0.465	0.0315	0.0315	0.465	0.465	0.0315	0.0315	0.465	0.465	0.0315
							+33.3	-33.3							
				-1.05	-15.50	-15.50	-1.05	+1.05	+15.50	+15.50	+1.05				
			-0.53				+0.53	-0.53				+0.53			
	+0.257	+0.257	+0.016	-0.017	-0.248	-0.248	-0.017	+0.017	+0.248	+0.248	+0.017	-0.017	-0.248	-0.248	-0.017
Σ	+0.257	+0.257	-0.514	-1.067	-15.748	-15.748	+32.763	-32.763	+15.748	+15.748	+1.067	+0.513	-0.248	-0.248	-0.017

FIG. 15 The 10th Story

$$FEM = \frac{1}{12} WL^2 = 33.3^{K\text{-}ft} \qquad K_1 = 3.48 \qquad K_2 = \frac{10800}{12 \times 12} = 75$$



AA'	AA"	AB
0.489	0.489	0.022
		+33.3
-16.30	-16.30	-0.76
		+0.37
-0.181	-0.181	-0.008
-16.481	-16.481	+32.962

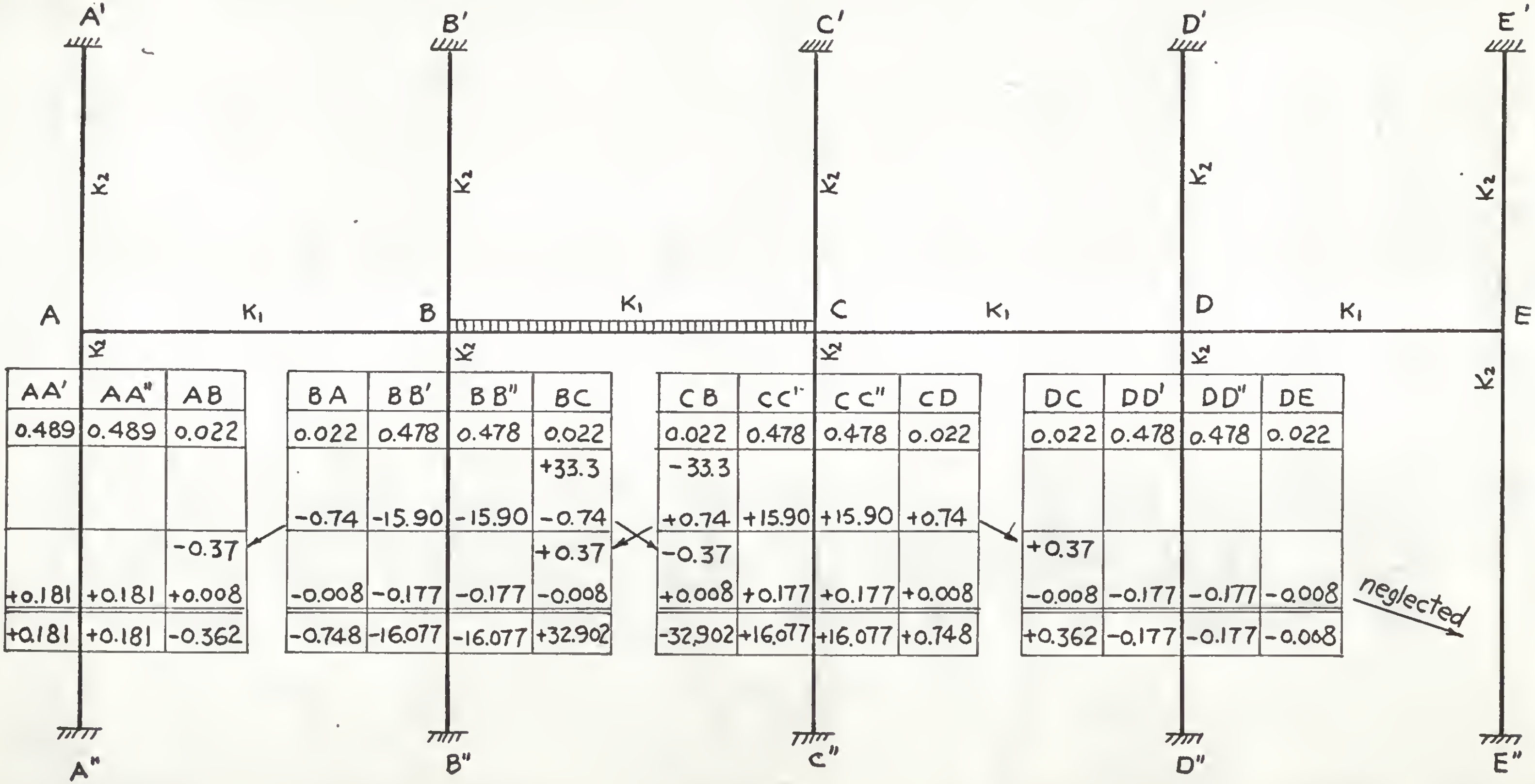
BA	BB'	BB"	BC
0.022	0.478	0.478	0.022
-33.3			
+0.74	+15.90	+15.90	+0.74
-0.378			
+0.008	+0.181	+0.181	+0.008
-32.93	+16.081	+16.081	+0.748

CB	CC'	CC"	CD
0.022	0.478	0.478	0.022
+0.37			
-0.008	-0.177	-0.177	-0.008
+0.362	-0.177	-0.177	-0.008

neglected

FIG. 16 The 2^{nd} Story

$$FEM = \frac{1}{12} WL^2 = 33.3^{K-ft} \qquad K_1 = 3.48 \qquad K_2 = 75$$



AA'	AA"	AB
0.489	0.489	0.022
		-0.37
+0.181	+0.181	+0.008
+0.181	+0.181	-0.362

BA	BB'	BB"	BC
0.022	0.478	0.478	0.022
			+33.3
-0.74	-15.90	-15.90	-0.74
			+0.37
-0.008	-0.177	-0.177	-0.008
-0.748	-16.077	-16.077	+32.902

CB	CC'	CC"	CD
0.022	0.478	0.478	0.022
-33.3			
+0.74	+15.90	+15.90	+0.74
-0.37			
+0.008	+0.177	+0.177	+0.008
-32.902	+16.077	+16.077	+0.748

DC	DD'	DD"	DE
0.022	0.478	0.478	0.022
+0.37			
-0.008	-0.177	-0.177	-0.008
+0.362	-0.177	-0.177	-0.008

FIG. 17 The 2^{nd} Story

Unit load on span	Column Moments					Girder Moments			
	$M_{AA'}$	$M_{BB'}$	$M_{CC'}$	$M_{DD'}$	$M_{EE'}$	M_{AB}	M_{BA}	M_{BC}	M_{CB}
AB	-24.30	-18.74	-2.54	-0.34	0	-24.29	-29.30	-10.52	-4.06
BC	-3.35	-17.97	-17.86	-2.34	0	-3.35	-9.87	-27.83	-27.92
CD	0	-2.43	-17.86	-17.97	-3.35	0	-1.45	-3.89	-10.65
DE	0	-0.34	-2.54	-18.74	-24.30	0	-0.202	-0.54	-1.44
All span	-20.95	-2,86	0	-2.86	-20.95	-20.91	-37.02	-35.00	-52.47

Table 2 Resulting Moments Caused by Unit Load on 40^{th} Story

Unit Load on Span	Column Moments						Girder Moments			
	$M_{AA'}$	$M_{AA''}$	$M_{BB'}$	$M_{BB''}$	$M_{CC'}$	$M_{CC''}$	M_{AB}	M_{BA}	M_{BC}	M_{CB}
AB	-15.54	-15.54	+14.06	+14.06	-0.68	-0.68	+31.08	-31.63	+3.51	+1.51
BC	+0.74	+0.74	-14.00	-14.00	+14.00	+14.00	-1.48	-3.50	+31.50	-31.50
CD	0	0	+0.68	+0.68	-14.00	-14.00	0	+0.68	-1.50	-3.50
DE	0	0	0	0	+0.68	+0.68	0	0	0	+0.17
All Span	-14.80	-14.80	+0.74	+0.74	0	0	+29.60	-34.45	+33.51	-33.33

Table 3 Resulting Moments Caused by Unit Load on 30th Story

Unit Load on Span	Column Moments						Girder Moments			
	$M_{AA'}$	$M_{AA''}$	$M_{BB'}$	$M_{BB''}$	$M_{CC'}$	$M_{CC''}$	M_{AB}	M_{BA}	M_{BC}	M_{CB}
AB	-16.15	-16.15	+15.40	+15.40	-0.38	-0.38	+32.30	-32.55	+1.75	+0.80
BC	+0.40	+0.40	-15.38	-15.38	+15.38	+15.38	-0.80	-1.74	+31.50	-31.50
CD	0	0	+0.38	+0.38	-15.38	-15.38	0	+0.04	-0.80	-1.74
DE	0	0	0	0	+0.38	+0.38	0	0	0	+0.04
All Spans	-15.75	-15.75	+0.40	+0.40	0	0	+31.50	-34.25	+32.45	-32.40

Table 4 Resulting Moments Caused by Unit Load on 20^{th} Story

Unit Load on Span	Column Moments						Girder Moments			
	$M_{AA'}$	$M_{AA''}$	$M_{BB'}$	$M_{BB''}$	$M_{CC'}$	$M_{CC''}$	M_{AB}	M_{BA}	M_{BC}	M_{CB}
AB	-16.36	-16.36	+15.76	+15.76	-0.25	-0.25	+32.72	-32.78	+1.07	+0.51
BC	+0.26	+0.26	-15.75	-15.75	+15.75	+15.75	-0.51	-1.07	+32.76	-32.76
CD	0	0	0.25	0.25	-15.75	-15.75	0	+0.02	-0.51	-1.07
DE	0	0	0	0	+0.25	+0.25	0	0	0	+0.02
All Spans	-16.10	-16.10	+0.26	+0.26	0	0	+32.21	-33.83	+33.32	-33.30

Table 5 Resulting Moments Caused by Unit Load on 10th Story

Unit Load on Span	Column Moments						Girder Moments			
	$M_{AA'}$	$M_{AA''}$	$M_{BB'}$	$M_{BB''}$	$M_{CC'}$	$M_{CC''}$	M_{AB}	M_{BA}	M_{BC}	M_{CB}
AB	-16.48	-16.48	+16.08	+16.08	-0.18	-0.18	+32.96	-32.93	+0.75	+0.36
BC	+0.18	+0.18	-16.08	-16.08	+16.08	+16.08	-0.36	-0.75	+32.90	-32.90
CD	0	0	+0.18	+0.18	-16.08	-16.08	0	+0.01	-0.36	-0.75
DE	0	0	0	0	+0.18	+0.18	0	0	0	+0.01
All Spans	-16.30	-16.30	+0.18	+0.18	0	0	+32.60	-33.67	+33.30	-33.00

Table 6 Resulting Moments Caused by Unit Load on 2nd Story

For 40th story (Table 2 and Fig. 4)

I. Max. girder moments

M_{BA}	D.L.	all span	$(2)(-37.92) = -75.80^{k-ft}$
	L.L.	on AB	$(0.5)(-29.30) = -14.65^{k-ft}$
	L.L.	on BC	$(0.5)(-9.87) = -4.98^{k-ft}$
	W.L.		$= -3.60^{k-ft}$
			-99.03^{k-ft}

II. Max. column moments

$M_{AA'}$	D.L.	all span	$(2)(-20.95) = -41.90^{k-ft}$
	L.L.	on AB	$(0.5)(-24.30) = -12.15^{k-ft}$
	W.L.		$= -3.60^{k-ft}$
			-57.65^{k-ft}

For 30th story (Table 3 and Fig. 4)

I. Max. girder moments

M_{BA}	D.L.	all span	$(3)(-34.45) = -103.35^{k-ft}$
	L.L.	on AB	$(1)(-31.65) = -31.65^{k-ft}$
	L.L.	on BC	$(1)(-3.50) = -3.50^{k-ft}$
	W.L.		$= -75.60^{k-ft}$
			-214.08^{k-ft}

II. Max. column moments

$M_{BB'}$	D.L.	all span	$(3)(+0.74) = +2.22^{k-ft}$
	L.L.	on AB and CD	$(1)(14.06+0.68) = +14.74^{k-ft}$
	W.L.		$= 79.20^{k-ft}$
			96.16^{k-ft}

For 20th story (Table 4 and Fig. 4)

I. Max. girder moments

M_{BA}	D.L.	all span	(3)(-34.25) = -102.75
	L.L.	on AB	(1)(-32.55) = - 32.55
	L.L.	on BC	(1)(- 1.74) = - 1.74
	W.L.		-147.60
			-284.64^{k-ft}

II. Max. column moments

$M_{BB'}$	D.L.	all span	(3)(+0.40) = + 1.20
	L.L.	on AB and CD	(1)(15.40+0.38) = +15.78
	W.L.		151.20
			-168.18^{k-ft}

For 10th story (Table 5 and Fig. 4)

I. Max. girder moments

M_{BA}	D.L.	all span	(3)(-33.83) = -101.49
	L.L.	on AB	(1)(-32.78) = - 32.78
	L.L.	on BC	(1)(- 1.07) = - 1.07
	W.L.		= -219.5
			354.84^{k-ft}

II. Max. column moments

$M_{BB'}$	D.L.	all span	(3)(+0.26) = + 0.78
	L.L.	on AB and CD	(1)(15.76+0.25) = + 16.01
	W.L.		= 223
			239.01^{k-ft}

For 2^{nd} story (Table 6 and Fig. 4)

I. Max. girder moments

M_{BA}	D.L.	all span	(3)(-33.67) =	-101.01
	L.L.	on AB	(1)(-32.93) =	- 32.93
	L.L.	on BC	(1)(- 0.75) =	- 0.75
	W.L.		=	-277.0
				-411.69^{k-ft}

II. Max. column moments

$M_{AA'}$	D.L.	all span	(3)(0.18) =	+ 0.54
	L.L.	on AB and CD	(1)(16.08+0.18) =	16.26
	W.L.		=	-280.0
				296.80^{k-ft}

(D) REVISION OF THE APPROXIMATE SECTIONS

From these combined moments caused by dead load, live load, and wind load, we may redesign the sections for girders and columns. By specification, working stresses can be increased 33.3 percent to 26,700 lbs per sq. in.

(1) For girders

For 40^{th} story

$M_{BA} = 99.03^{k-ft} \times 12,000 = 1,190,000^{lb-in}$

$S = 1,190,000/26,000 = 44.6^{in^3}$

Use 14WF43 ($S = 62.7^{in^3}$ $\quad I = 429.0^{in^4}$)

For 30^{th} story

$M_{BA} = 214.08 \times 12,000 = 2,570,000^{lb-in}$

$S = 2,570,000/26,700 = 96.5^{in^3}$

Use 14WF68 ($S = 103^{in^3}$ $\quad I = 724.1^{in^4}$)

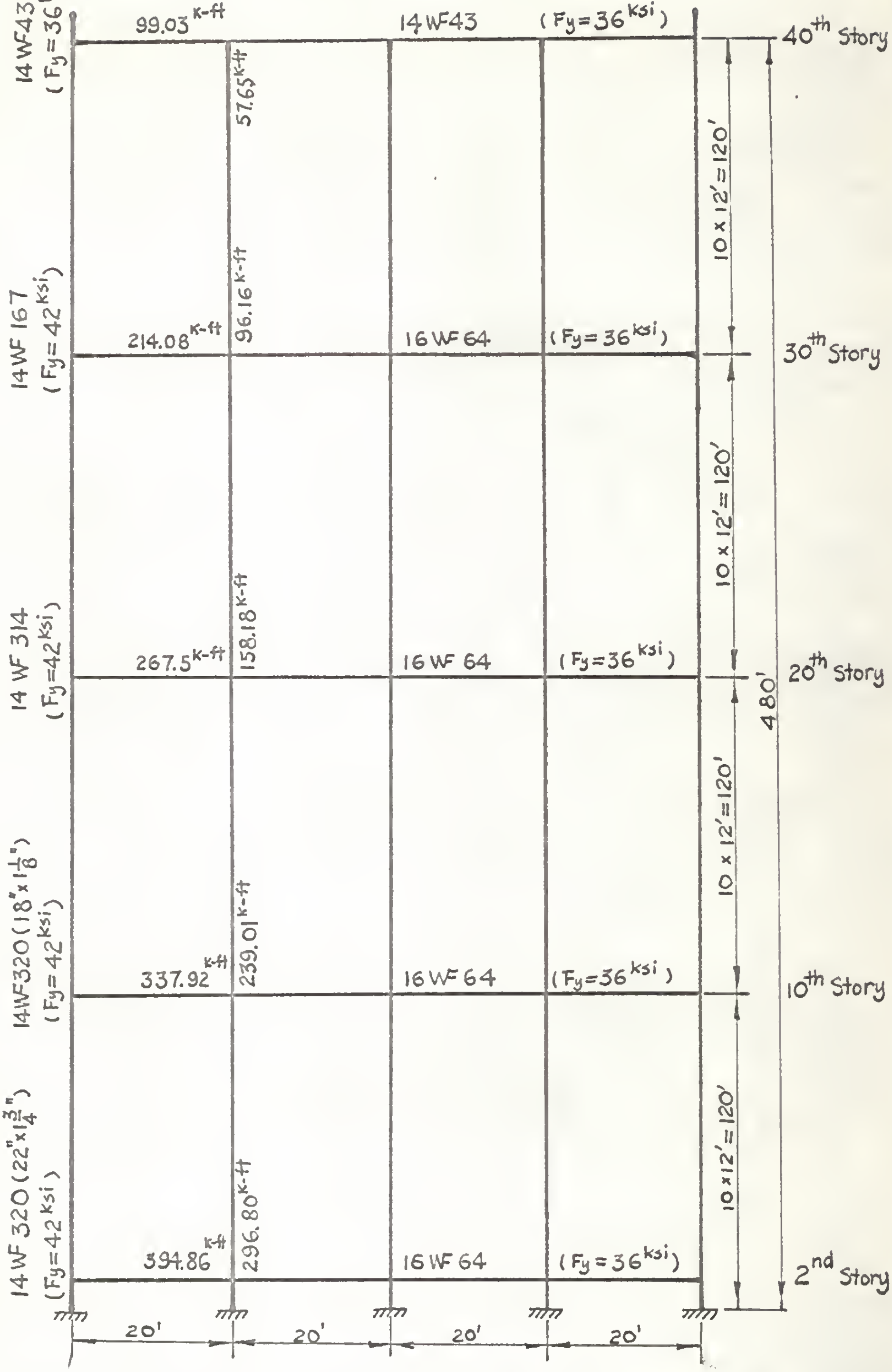


FIG. 18 Max. Girder and Column Moments Due to Dead, Live, and Wind Loads and the Original Approximate Sizes in Girders and Columns

For 20^{th} story

$M_{BA} = 284.64 \times 12,000 = 3,420,000^{lb\text{-}in}$

$S = 3,420,000/26,700 = 128^{in^3}$

Use 14WF84 ($S = 130.9^{in^3}$ $I = 928.4^{in^4}$)

For 10^{th} story

$M_{BA} = 354.84 \times 12,000 = 4,260,000^{lb\text{-}in}$

$S = 4,260,000/26,700 = 160^{in^3}$

Use 14WF103 ($S = 163^{in^3}$ $I = 1165.8^{in^4}$)

For 2^{nd} story

$M_{BA} = 411.69 \times 12,000 = 4,930,000^{lb\text{-}in}$

$S = 4,930,000/26,700 = 185^{in^3}$

Use 14WF119 ($S = 189.4^{in^3}$ $I = 1373.1^{in^4}$)

(2) For columns

The choice must be made for resisting axial stresses and moments caused by dead, live and wind loads with normal working stresses increased 33.3 percent. Also, members must be designed to satisfy the normal allowable stresses for pure gravity loads. Therefore, there are two cases to be considered.

Case I-----D.L. + L.L.

(a) Axial force

(b) Bending moment-----Maximum at $M_{AA'}$

At normal working stress of

F_a = varies with $K\frac{l}{r}$

$F_b = 20^{ksi}$

Case II-----(D.L. + L.L.) + W.L.

(a) Axial force

(b) Bending moment-----Maximum at $M_{BB'}$

At increasing one-third normal working stress of

F_a = increasing F_a in case I

$F_b = 20 \times 1.33 = 26.7^{ksi}$

From AISC Specification of 1963, Sixth Edition, there are two formulas which must be satisfied

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{when } \frac{f_a}{F_a} \leq 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{(1 - \frac{f_a}{F_a})} \frac{f_b}{F_b} \leq 1.0 \qquad \text{when } \frac{f_a}{F_a} \geq 0.15$$

Where

$f_a = \frac{P}{A}$ = actual axial unit stress

$f_b = \frac{M}{S}$ = actual bending unit stress

F_a = axial compression stress permitted in the absence of bending stress.

F_b = bending stress permitted in the absence of axial stress.

K = effective length factor, in this problem, it is 1.0 from the AISC Specification, Section 1.8 page 5-117.

r = radius of gyration

The column subjected to both axial compression and bending stresses shall be proportioned to satisfy the above formulas in the above two cases.

For 40^{th} story

Case I-----D.L. + L.L.

(a) Axial force (Table 1) 50^k

(b) Bending moment (Table 2)

$M_{AA'}$ D.L. all span $(2)(-20.95) = -41.90^{k\text{-}ft}$

L.L. on AB $(0.5)(-24.30) = -12.15^{k\text{-}ft}$

$54.05^{k\text{-}ft}$

For 14WF43 $F_y = 36^{ksi}$ $(A = 12.66^{in^2} \quad S = 62.7^{in^3})$

$$f_a = \frac{P}{A} = \frac{50}{12.66} = 3.95^{ksi}$$

$$f_b = \frac{M}{S} = \frac{54.05 \times 12}{62.7} = 10.3^{ksi}$$

$$\frac{l}{r} = \frac{12 \times 12}{1.89} = 76$$

$F_a = 15.79^{ksi}$ (AISC p. 5-68)

$F_b = 20^{ksi}$

$$\frac{f_a}{F_a} = \frac{3.95}{15.79} = 0.25 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.25}\frac{f_b}{F_b} = 0.25 + 1.13\,\frac{10.3}{20} = 0.25 + 0.583 = 0.83 < 1 \quad \text{O.K.}$$

Case II------(D.L. + L.L.) + W.L.

(a) Axial (Table 1 and Fig. 5) 50.36^{k}

(b) Bending moment (from Max. moment $M_{BB'}$) $61.25^{k\text{-}ft}$

$$f_a = \frac{50.36}{12.36} = 3.98^{ksi}$$

$$f_b = \frac{61.25 \times 12}{62.7} = 11.7^{ksi}$$

$$\frac{f_a}{F_a} = \frac{3.98}{15.79 \times 1.33} = 0.19 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.19}\frac{f_b}{F_b} = 0.19 + 1.05\,\frac{11.7}{26.7} = 0.19 + 0.30 = 0.65 < 1 \quad \text{O.K.}$$

For 30^{th} story

Case I-----D.L. + L.L.

(a) Axial force (Table 1) 770^{k}

(b) Bending moment (Table 3)

$M_{AA'}$ D.L. all span $(3)(-14.80) = -44.40^{k-ft}$

L.L. on AB $(1)(-15.54) = -15.54^{k-ft}$

59.94^{k-ft}

For 14WF167 $F_y = 42^{ksi}$ $(A = 49.09^{in^2}$ $S = 267.3^{in^3}$ $r = 4.01)$

$$f_a = \frac{770}{49.09} = 15.7^{ksi}$$

$$f_b = \frac{59.94 \times 12}{267.3} = 2.69^{ksi}$$

$$K\frac{l}{r} = \frac{144}{4.01} = 35.9$$

$F_a = 22.50$ (AISC Code p. 5-78)

$$\frac{f_a}{F_a} = \frac{15.7}{22.50} = 0.70 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.70}\frac{f_b}{F_b} = 0.70 + 2.84\frac{2.69}{20} = 0.70 + 0.38 = 1.08 > 1$$

Overstress in vertical loading in normal working stress.

Try 14WF176 $F_y = 42^{ksi}$ $(A = 51.73^{in^2}$ $S = 281.9^{in^3}$ $r = 4.02)$

$$f_a = \frac{770}{51.73} = 14.9^{ksi}$$

$$f_b = \frac{59.94 \times 12}{281.9} = 2.56^{ksi}$$

$$K\frac{l}{r} = \frac{144}{4.02} = 35.9 \qquad F_a = 22.5$$

$$\frac{f_a}{F_a} = \frac{14.9}{22.5} = 0.661 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.661}\frac{f_b}{F_b} = 0.661 + 2.51\frac{2.56}{20} = 0.982 < 1.0 \qquad \text{O.K.}$$

Case II-----(D.L. + L.L.) + W.L.

(a) Axial force (Table 1 and Fig. 5) 777.56^k

(b) Bending moment $M_{BB'}$ $105.48^{k\text{-}ft}$

$$f_a = \frac{777.56}{51.73} = 15^{ksi}$$

$$f_b = \frac{105.48 \times 12}{281.9} = 4.5^{ksi}$$

$$\frac{f_a}{F_a} = \frac{15}{22.5 \times 1.33} = 0.5 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.5}\frac{f_b}{F_b} = 0.5 + 1.7\frac{4.5}{26.7} = 0.786 < 1.0 \qquad \text{O.K.}$$

For 20th story

Case I-----D.L. + L.L.

(a) Axial force (Table 1) 1490^k

(b) Bending moment (Table 4)

$M_{AA'}$	D.L.	all span	$(3)(-15.75) = -47.25^{k\text{-}ft}$
	L.L.	on AB	$(1)(-16.15) = -16.15^{k\text{-}ft}$
			$63.40^{k\text{-}ft}$

For 14WF314 $F_y = 42^{ksi}$ $(A = 92.30^{in^2}$ $S = 511.9^{in^3}$ $r = 4.20)$

$$f_a = \frac{1490}{92.30} = 16.15^{ksi}$$

$$f_b = \frac{63.40 \times 12}{511.9} = 1.48^{ksi}$$

$$K\frac{l}{r} = \frac{144}{4.20} = 34.3$$

$$F_a = 22.66$$

$$\frac{f_a}{F_a} = \frac{16.15}{22.66} = 0.71 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.71}\ \frac{f_b}{F_b} = 0.71 + 2.93\ \frac{1.48}{20} = 0.93 < 1.0$$ O. K.

Case II-----(D.L. + L.L.) + W.L.

(a) Axial force (Table 1 and Fig. 5) 1504.76^{k}

(b) Bending moment $M_{BB'}$ $158.18^{k\text{-}ft}$

$$f_a = \frac{1504}{92.3} = 16.3^{ksi}$$

$$f_b = \frac{168.18x12}{511.9} = 3.95^{ksi}$$

$$\frac{f_a}{F_a} = \frac{16.3}{22.66x1.33} = 0.541 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.541}\ \frac{f_b}{F_b} = 0.541 + 1.85\ \frac{3.95}{26.7} = 0.80 < 1.0$$ O. K.

For 10^{th} story

Case I-----D.L. + L.L.

(a) Axial force (Table 1) 2210^{k}

(b) Bending moment (Table 5)

$M_{AA'}$	D.L.	all span	$(3)(-16.10) = -48.30^{k\text{-}ft}$
	L.L.	on AB	$(1)(-16.36) = -16.36^{k\text{-}ft}$
			$64.66^{k\text{-}ft}$

For 14WF320 (18" x 1 1/8") $F_y = 42^{ksi}$

(A = 134.2^{in^2} S = 777 r = 4.5)

$$f_a = \frac{2210}{134.2} = 16.4^{ksi}$$

$$f_b = \frac{64.66x12}{777} = 1.0^{ksi}$$

$$K\frac{l}{r} = \frac{144}{4.5} = 32$$

$$F_a = 22.88$$

$$\frac{f_a}{F_a} = \frac{16.4}{22.88} = 0.716 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.716}\frac{f_b}{F_b} = 0.716 + 2.99\frac{1}{20} = 0.865 < 1.0 \qquad \text{O.K.}$$

Case II-----(D.L. + L.L.) + W.L.

(a) Axial force (Table 1 and Fig. 5) 2331.95^k

(b) Bending moment $M_{BB'}$ 239.01^{k-ft}

$$f_a = \frac{2331.95}{134.6} = 17.3^{ksi}$$

$$f_b = \frac{239.01x12}{777} = 3.69^{ksi}$$

$$\frac{f_a}{F_a} = \frac{17.3}{22.88x1.33} = 0.57 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.57}\frac{f_b}{F_b} = 0.57 + 1.97\frac{3.69}{26.7} = 0.84 < 1.0 \qquad \text{O.K.}$$

For 2^{nd} story

Case I-----D.L. + L.L.

(a) Axial force (Table 1) 2786^{k}

(b) Bending moment (Table 6)

$M_{AA'}$ D.L. all span $(3)(-16.30) = -48.90^{k\text{-}ft}$

L.L. on AB $(1)(-16.48) = -16.48^{k\text{-}ft}$

$65.38^{k\text{-}ft}$

For 14WF (22" x 1 3/4") $F_y = 42^{ksi}$

$(A = 171.1^{in^2} \quad S = 1063^{in^3} \quad r = 5.26)$

$$f_a = \frac{2786}{171.1} = 16.3^{ksi}$$

$$f_b = \frac{65.38x12}{1063} = 0.734^{ksi}$$

$$K\frac{l}{r} = \frac{144}{5.26} = 27.4 \qquad F_a = 23.30^{ksi}$$

$$\frac{f_a}{F_a} = \frac{16.3}{23.30} = 0.70 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.7}\frac{f_b}{F_b} = 0.7 + 2.83\,\frac{0.734}{20} = 0.804 < 1.0 \qquad \text{O. K.}$$

Case II-----(D.L. + L.L.) + W.L.

(a) Axial force (Table 1 and Fig. 5) 2813.7^{k}

(b) Bending moment $M_{BB'}$ $296.80^{k\text{-}ft}$

$$f_a = \frac{2813.7}{171.1} = 16.48^{ksi}$$

$$f_b = \frac{296x12}{1063} = 3.35^{ksi}$$

$$\frac{f_a}{f_b} = \frac{16.48}{23.3 \times 1.33} = 0.53 > 0.15$$

$$\frac{f_a}{F_a} + \frac{0.85}{1-0.53}\frac{f_b}{F_b} = 0.53 + 1.81\,\frac{3.35}{26.7} = 0.787 < 1.0$$ O. K.

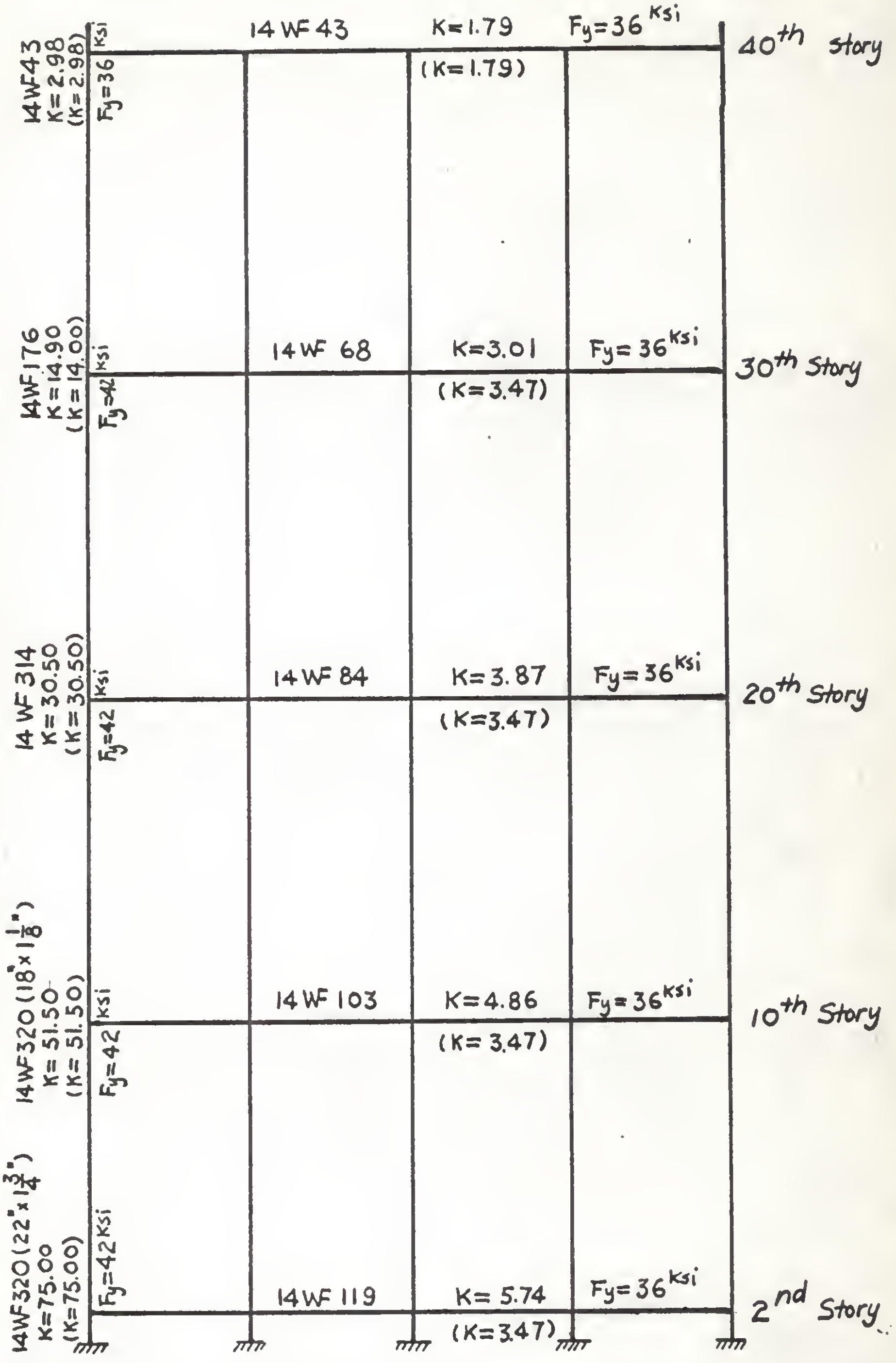


FIG. 19 Revised Size of Girders and Columns by Second Analysis. Stiffness Factors of Approximate Section Are Shown in Parenthesis

FINAL DESIGN OF THE BUILDING

After obtaining the new sizes in columns and girders, we can revise the moments of the frame by the moment distribution method using the new stiffness factors if necessary.

These moments and axial forces in the typical floors are the design factors for the whole building. Moments and axial forces in other floors can be obtained by plotting the story moment and axial force curves. These curves turn out to be nearly a straight line. One can easily interpolate the design data from the curves and obtain the desired sizes of columns and girders by the equations from the handbook.

For the purpose of practical design, the same size can be used for more than two stories.

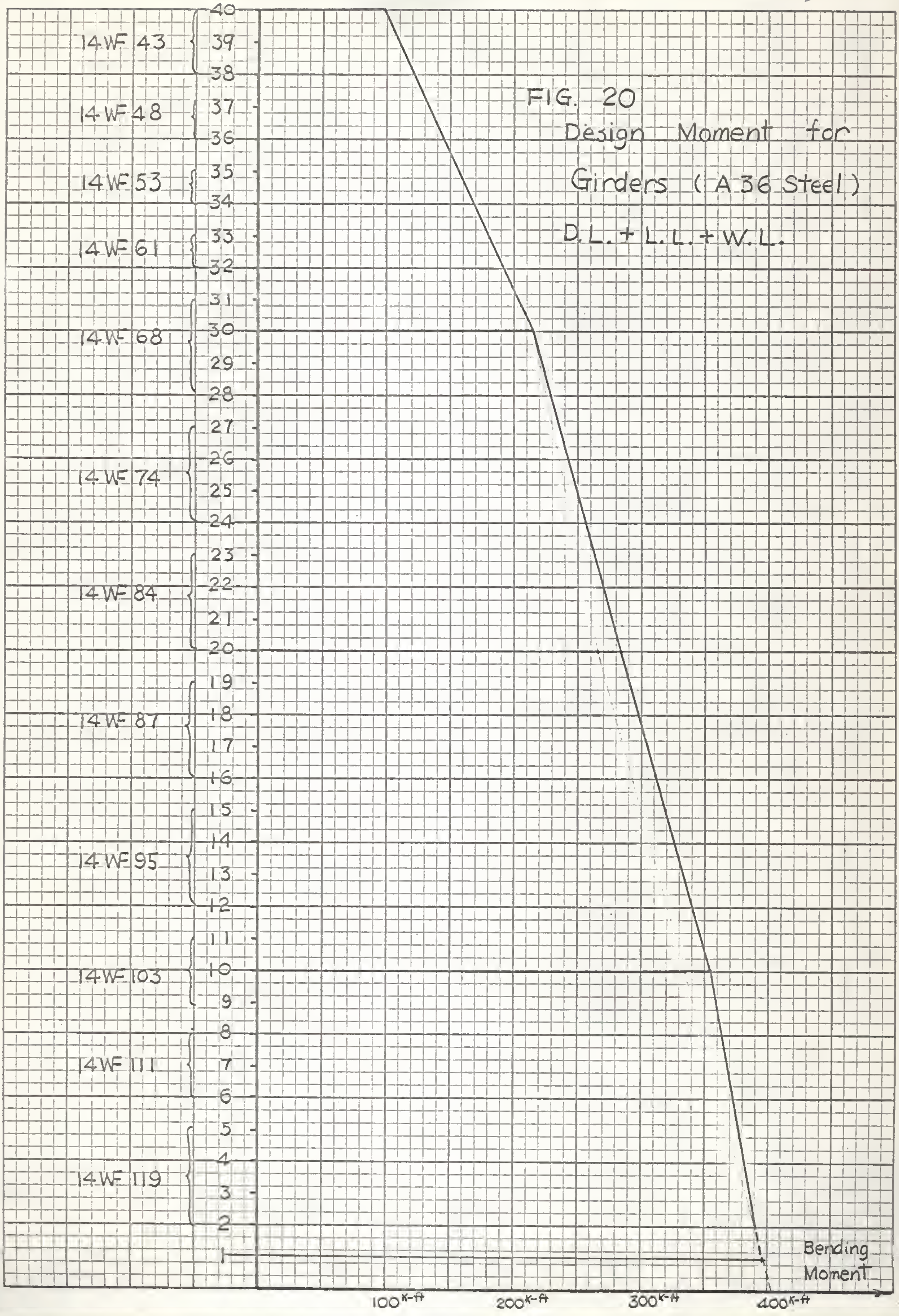


FIG. 20
Design Moment for
Girders (A 36 Steel)
D.L. + L.L. + W.L.
14 WF 43
14 WF 48
14 WF 53
14 WF 61
14 WF 68
14 WF 74
14 WF 84
14 WF 87
14 WF 95
14 WF 103
14 WF 111
14 WF 119
40
39
38
37
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
100 K-ft
200 K-ft
300 K-ft
400 K-ft
Bending Moment

53

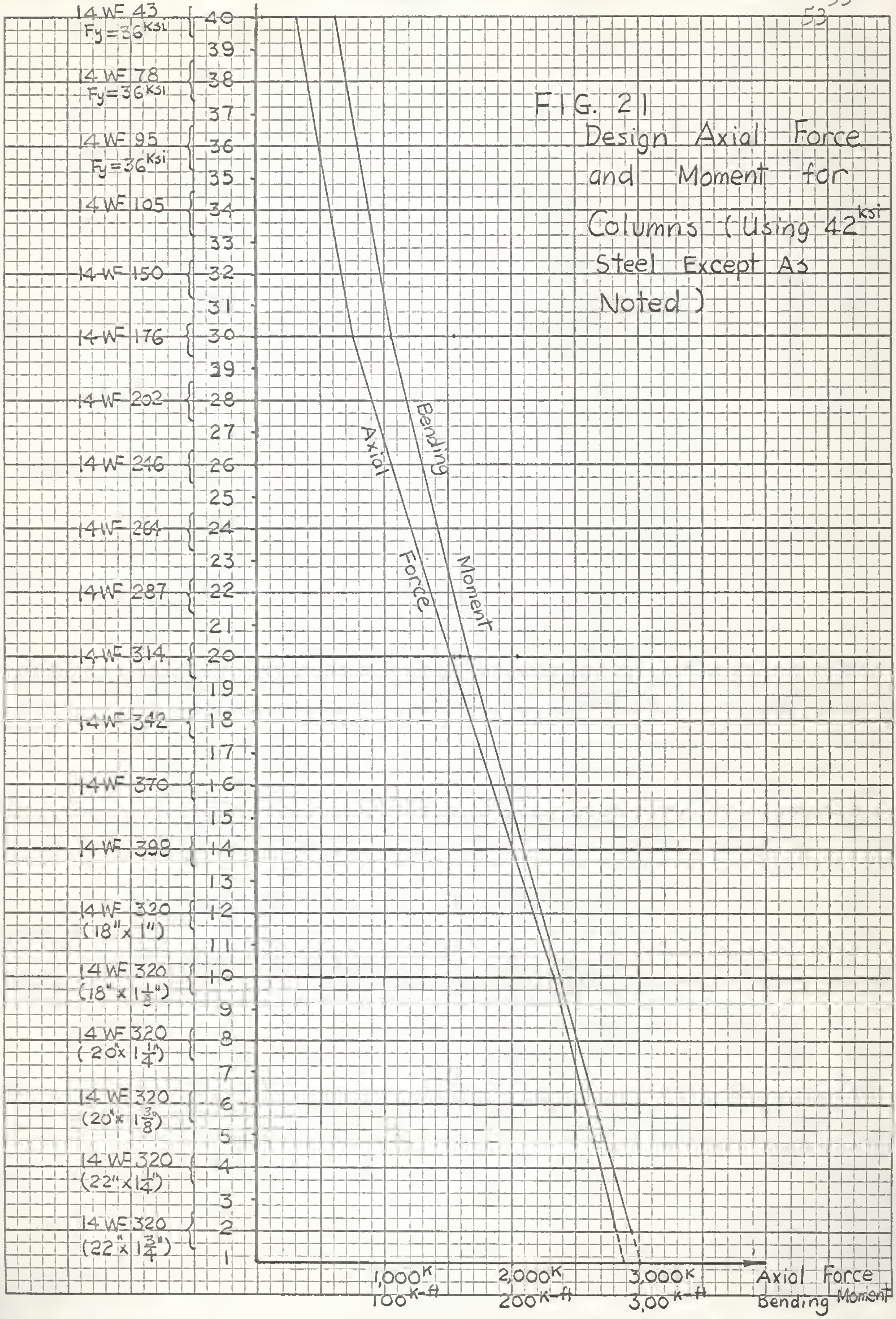


FIG. 21
Design Axial Force and Moment for Columns (Using 42 ksi Steel Except As Noted)

CONCLUSION

Among the many complicated structural analyses is the study of wind stresses in tall buildings. The exact analysis is impossible because of many uncertainties involved as mentioned in the INTRODUCTION. Approximate analyses by the moment distribution method is not convenient and by the slope deflection method requires the knowledge and availability of an electronic computer and its operation. The practicing structural engineer must resort to approximate methods of analysis in order to obtain a solution within a reasonable time. In this report, we break the whole building frame into some typical subframes. After obtaining the moments in each typical story, we plot the moments for the typical stories and using straight line interpolation obtain the moments on each story. Then we design each story by using these moments. This systematic method of breaking up the whole frame into subframes is very convenient in the analysis of tall buildings.

ACKNOWLEDGMENTS

The author is greatly indebted to his major instructor, Professor Reed F. Morse, for his aid and criticism during the preparation of this report.

BIBLIOGRAPHY

1. Cross, H. and Morgan, N. D.
   Continuous Frames of Reinforced Concrete. New York: John Wiley and Sons, 1932. pp 214-238.

2. Fleming, R.
   Wind Stress in Building. New York: John Wiley and Sons, 1930.

3. Grinter, L. E.
   Design of Modern Steel Structures. New York: Macmillan Company, 1941. pp 353-384.

4. Maugh, L. C.
   Statically Indeterminate Structures. New York: John Wiley and Sons, 1947.

5. Michalos, J.
   Theory of Structural Analysis and Design. New York: Ronald Press Co., 1958.

6. Spur, H. V.
   Wind Bracing. New York: McGraw-Hill Book Co., 1930.

7. Sutherland, H. and Bowman, H. L.
   Structural Theory. New York: John Wiley and Sons, 4th. Ed., 1950. pp 291-304.

8. Urquhart, L. C. and O'Rourke, C. E.
   Design of Concrete Structures. New York: McGraw-Hill Book Co., 5th Ed., 1954. pp 173-206.

WIND STRESS ANALYSIS IN TALL BUILDINGS

by

JOSEPH CHI-KIT LEE

B. S., Cheng Kung University, 1958
Tainan, Taiwan, China

AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1964

The analysis of stresses in tall buildings due to vertical and wind loads is tedious and time consuming by any method. An exact analysis is impossible because of many uncertainties involved such as the exact distribution of wind pressure on the building, the torsion of the tall frame, and the secondary moments. Approximate analysis by the moment distribution method is not convenient and by the slope deflection method requires the knowledge and availability of an electronic computer and its operations. In order to obtain a solution within a reasonable time, the writer presents here a speedy and modified combination of several methods to solve such a problem.

First, the cantilever method is used to solve for the wind stresses which are combined with the stresses due to vertical loads in order to get the approximate sections for columns and girders. Second, both the portal method and a modified moment distribution method are used. The portal method is used for a more refined analysis of the wind stresses. The modified moment distribution method which requires the approximate stiffness factors of all members, and breaks up the whole building frame into some typical subframes, is used to recalculate the stresses due to vertical loads. After obtaining the combination of these two kinds of stresses in the typical stories, curves for the moments in the girders and columns of the typical stories are plotted and straight line interpolation is used to obtain the moments on each story. Then, by using these moments, the approximate sections on the typical stories can be revised and the sections on other stories can therefore be designed. This systematic method of breaking up the whole frame into subframes is very convenient in the analysis of tall buildings.